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Storage/Sedimentation Facilities for Control of Storm and Combined Sewer Overflows

Design Manual

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NOTICE

The U.S. Environmental Protection Agency (EPA) through its Office of Research and Development funded and managed the preparation of this document under Contract 68-Ø3-2877 with Metcalf & Eddy, Inc. The draft report was prepared in the time frame of September 1979 to October 1981 and was revised in 1997 for this publication. Although the report was prepared many years ago, it was not published at that time. It is being released currently in order to provide information to communities in support of their wet-weather flow management efforts. Despite the report's revision, some of its content may no longer be entirely current. The document has been subjected to the Agency's peer and administrative review, and it has been approved for publication as an EPA document. Mention of trade names or commercial products does not constitute endorsement or recommendation for use.

FOREWORD

The U.S. Environmental Protection Agency is charged by Congress with protecting the Nation's land, air, and water resources. Under a mandate of national environmental laws, the Agency strives to formulate and implement actions leading to a compatible balance between human activities and the ability of natural systems to support and nurture life. To meet this mandate, EPA's research program is providing data and technical support for solving environmental problems today and building a science knowledge base necessary to manage our ecological resources wisely, understand how pollutants affect our health, and prevent or reduce environmental risks in the future.

The National Risk Management Research Laboratory is the Agency's center for investigation of technological and management approaches for reducing risks from threats to human health and the environment. The focus of the Laboratory's research program is on methods for the prevention and control of pollution to air, land, water, and subsurface resources; protection of water quality in public water systems; remediation of contaminated sites and ground water; and prevention and control of indoor air pollution. The goal of this research effort is to catalyze development and implementation of innovative, cost-effective environmental technologies; develop scientific and engineering information needed by EPA to support regulatory and policy decisions; and provide technical support and information transfer to ensure effective implementation of environmental regulations and strategies.

This publication has been produced as part of the Laboratory's strategic long-term research plan. It is published and made available by EPA's Office of Research and Development to assist the user community and to link researchers with their clients.

E. Timothy Oppelt, Director
National Risk Management Research Laboratory

ABSTRACT

This report describes applications of storage facilities in wet-weather flow management and presents step-by-step procedures for the analysis and design of storage-treatment facilities. Retention, detention, and sedimentation storage are classified and described. International as well as national state-of-the-art projects are discussed.

Retention storage facilities capture and dispose of stormwater runoff through infiltration, percolation, and evaporation. Detention storage is temporary storage for stormwater runoff or combined sewer overflow. Stored flows are subsequently returned to the sewerage system at a reduced rate of flow when downstream capacity is available, or the flows are discharged to the receiving water with or without further treatment. Sedimentation storage alters the wastewater stream by gravity separation. The stormwater runoff and combined sewer overflow must be characterized to estimate the efficiency of any sedimentation basin.

The detailed design methodology of the storage and/or sedimentation facility presented in this report includes: identifying functional requirements; identifying site constraints; establishing basis of design; selecting storage and/or treatment option; and conducting a cost analysis.

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Section 1

INTRODUCTION AND USERS' GUIDE

As municipal wastewater treatment is upgraded in accordance with the Federal Clean Water Act, urban stormwater runoff and combined sewer overflows (CSO) are emerging as significant sources of surface water pollution in the United States. A 1975 survey of 56 public agencies located throughout the United States revealed that "...control [of] stormwater pollution from sources other than erosion to make significant improvements in existing wet-weather quality of streams, lakes, etc." ranked second only to flood control as a stormwater management goal [1].

Temporary storage of runoff, a widely used method of flood control, is gaining acceptance in the United States as a cost-effective means of reducing the pollutant load of stormwater runoff. Existing flood control facilities may be retrofitted and new flood control facilities designed to enhance pollution removal. Similarly, the basic reason for incorporating storage into CSO/urban runoff control systems is to provide flow equalization so the overall cost of the storage-treatment system can be optimized. Flow equalization at the treatment plant and combined storage and/or sedimentation effects have been demonstrated as cost-effective measures to reduce pollution from CSO systems.

The purpose of this design manual is to summarize applications of storage facilities in stormwater management and to present step-by-step procedures for analysis and design of stormwater and CSO storage and/or sedimentation treatment facilities. The manual is directed toward the technical designer and planner and primary emphasis is placed on the cost-effective reduction of total pollutants discharged.

STORMWATER, URBANIZATION, AND CONTROL

Urban stormwater runoff is not a new problem. Among the earliest examples of public works are urban drainage systems. The early sewers were designed to minimize flooding by rapidly transporting converging runoff through developed low-lying areas. The stormwater was discharged to natural drainage channels or streams with little regard to downstream effects.

More recently, it has been recognized that urban areas have significant impacts on stormwater runoff quantity and quality and on the attainment of downstream receiving water quality objectives. The response of a watershed to precipitation for undeveloped conditions and for urbanized conditions with and without stormwater controls is illustrated in Figure 1.

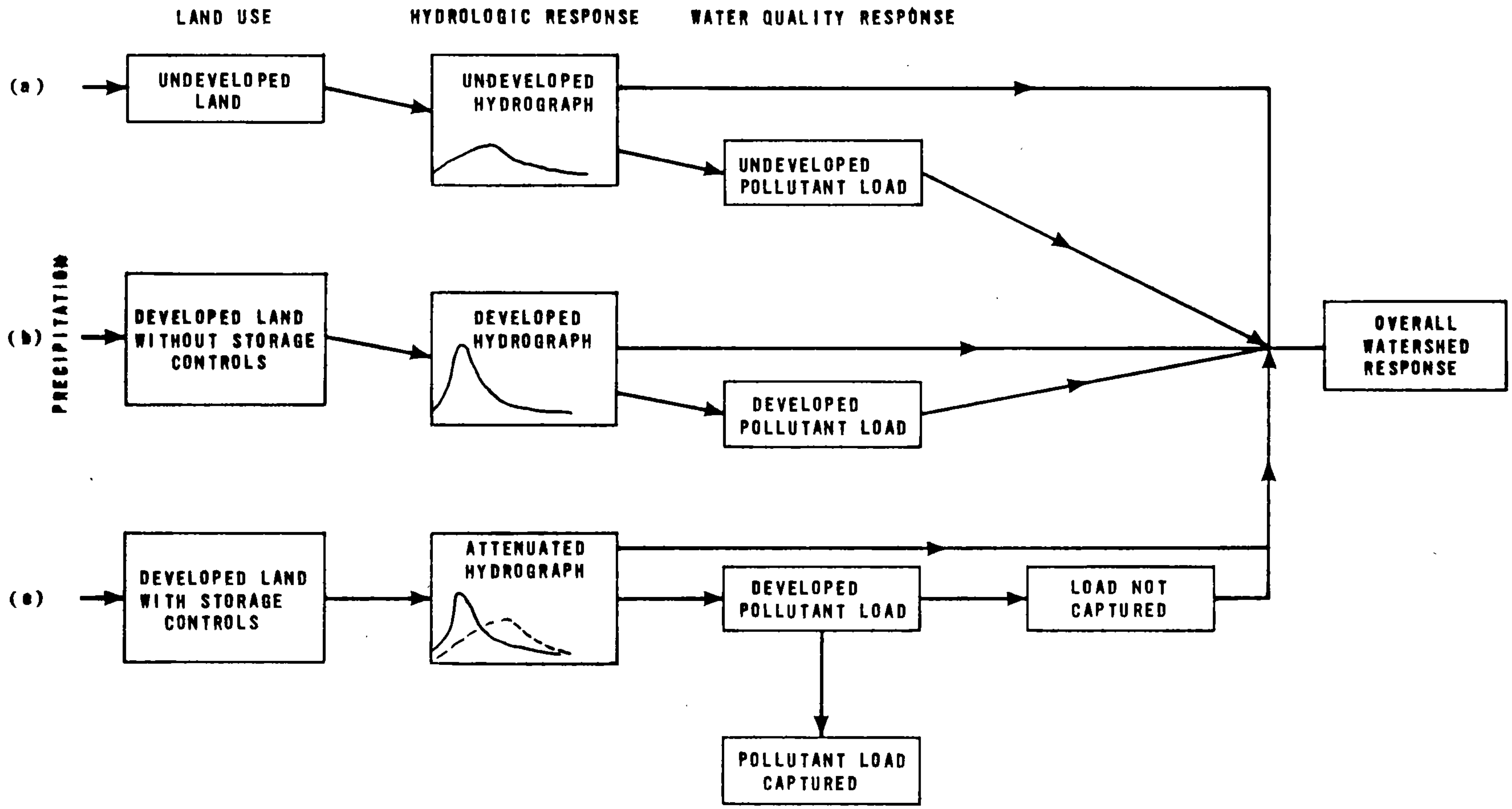

Figure 1. Response of watershed to precipitation under different conditions [2].

In the figure, hydrographs show the rate of flow of runoff (y-axis) at a given point versus time (x-axis). Under natural conditions, rain falling on the ground surface may do one of three things: it may be intercepted and held on vegetation, roots, and the ground as the surfaces are wetted; it may infiltrate through the ground surface and percolate downward to become part of the groundwater; or it may collect on the surface in depressions or move across the surface as runoff. The paved areas and buildings that characterize an urban environment prevent or retard infiltration. Urban areas usually have much less vegetation so that interception is reduced and heavily trafficked ground surfaces compact and become less pervious.

Precipitation, falling through the air and flowing over the ground surface, captures, dissolves, and suspends a portion of the material contacted and carries these "pollutants" along, usually to a receiving water. Densely populated areas are characterized by the discharge of waste materials to the air and ground surface as well as to water bodies. Runoff from urban areas is contaminated by this waste material. For example, it may contain three to four times the concentration of suspended material as is typically found in raw domestic wastewater as well as significant quantities of toxic and oxygen demanding substances, nutrients, pesticides, salts, and bacteria.

In addition to the pollution content of urban runoff itself, the runoff in many older cities of the United States is combined with municipal wastewater in the sewer system. Sewers were originally constructed for stormwater conveyance. When human and industrial wastes came to be recognized as urban problems in the mid and late 19th century, in many cities these wastes were introduced into the storm sewer system making it a combined sewer system. When overflows of combined sewers occur, a mixture of runoff and raw municipal wastewater is spilled. A typical combined sewer system is illustrated in Figure 2. In 1995, the EPA estimated the CSO abatement costs for the 1100 communities served by combined sewer systems to be over $40 billion. [3a]

The cost of controlling stormwater pollution can be substantial. The American Public Works Association's 1992 report, Nationwide Costs to Implement BMPs [3b] identified possible capital costs of up to $407 billion and operation and maintenance costs of $542 billion/year to meet water quality standards for stormwater discharges. While legislative oversight reviews and reassessments may reduce or defer elements of the program, the development and application of cost-effective control technologies remains an essential national goal.

The major problem in controlling stormwater runoff is its variability. Rainfall and runoff occur as intermittent and random events, varying widely in volume and rate of flow during single events and from one event to the next. Transport and treatment facilities for excess wet-weather flow control, sized to handle some medium storm size, frequently are idle during dry periods and overflow during large storms. Secondary (dual) use of facilities during nonstorm periods may improve their cost effectiveness and should be evaluated on an integrated total system basis.

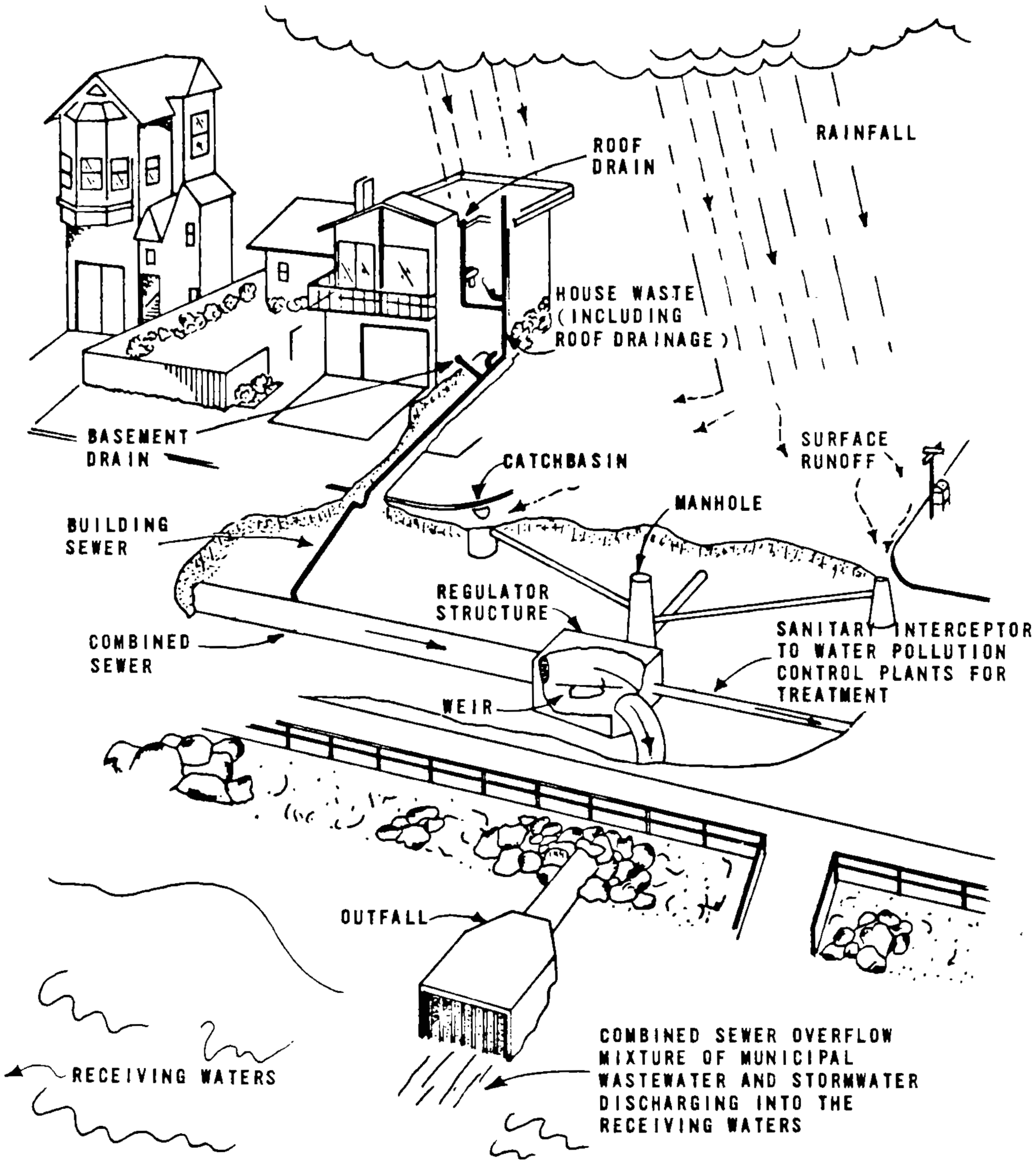

Figure 2. Common elements of a combined sewer system [4].

Temporary storage of excess runoff can be an effective and economical method of controlling stormwater flooding and pollution. Storage capacity can reduce flowrates and enhance pollutant removals. Excess runoff stored during large storms or during more intense rainfall periods can be released slowly when capacity in the drainage and treatment system or stream channel is available. Because the peak rate of flow is much less, smaller capacities are needed to transport and to treat the same quantity of flow, and overflows or flooding occur less frequently. In some CSO systems in Europe, selective storage timed to peak pollutant loads has been effectively implemented with resultant high pollutant capture.

When the storage capacity is exceeded, storage basins may provide treatment to the overflow by sedimentation. The ability of flowing water to carry heavier solid materials is directly related to the velocity of flow. In traveling through a storage facility, the stormwater slows, so that the heavier solids are deposited. By carefully designing inlet and outlet structures to maximize the sedimentation effect, and providing some method of removing and disposing of captured solids, significant water quality benefits can be achieved.

PURPOSE OF THE MANUAL

Over the past decade, there has been a large commitment by the U.S. Environmental Protection Agency (USEPA) to identify pollution sources other than municipal wastewater discharges and to develop viable methods for their control. A large amount of information on stormwater runoff and, particularly, CSO characteristics and treatabilities, has been developed. Much less information is available on the effect of stormwater runoff on receiving water impacts. The purpose of the design manual is to summarize the existing information for stormwater and CSO storage and/or sedimentation control facilities, including examples of foreign practice, and to suggest step-by-step analysis and design procedures for their application.

ORGANIZATION

For ease of reference and consistency, certain terminology and classifications of control systems have been adopted for this manual (see Section 3). Basic categories are: (1) the type of collection system, and (2) the general placement of the storage and/or sedimentation facility or practice within the collection network. The user should recognize that some devices, theories, or practices will fall in more than one category. In these cases, the detailed discussion is presented in the category representing the most common use and cross referenced under other categories. For example, sedimentation theory is described in connection with downstream controls (where treatment impacts are paramount) and infiltration/percolation is described under source controls (where maximum system sizing economies are likely to be achieved). Fundamentals on storage volume requirements, common to all applications, are introduced under "system planning" and elaborated upon where applicable under the specific control description.

Typically in stormwater management systems, problems and remedial costs increase as one moves downstream in the collection network. Also, there tends to be a direct impact on the design of downstream facilities as a consequence of actions or lack of actions in terms of upstream controls, whereas the reverse is not the case. The order of presentation of the design procedures and the design criteria starts with facilities for combined sewer systems and then proceeds to separate storm sewer systems. Facilities for detention, sedimentation, and retention are presented in that order.

USERS' GUIDE

This manual is organized to present, in a logical sequence, specific application methods and design procedures for storage and/or sedimentation control of stormwater pollution. It is, however, a design manual, and sequential reading of the material contained herein is not necessary. As an aid to ready use of the manual, the following description of section contents is provided.

Section 1 - Introduction and Users' Guide

Introduction. The problems of flooding, groundwater loss, and water pollution that may result from urban stormwater and CSO are briefly introduced. The importance of stormwater as an urban pollution source is emphasized. The potential of storage and/or sedimentation as a control method is discussed.

Purpose of the Manual. The specific aims of the design manual are given.

Organization. The classification scheme upon which the manual is structured is described.

Users' Guide. A brief summary of the purpose, content, and organization of each of the chapters is presented as a quick reference for the potential user of the manual.

Section 2 - Urban Stormwater: An Overview

Urban Stormwater Pollution. The sources and representative characteristics of urban stormwater pollution are described; the concept of "first flush" is introduced; and problem intensification through urban development is illustrated by example. The approach to receiving water protection and restoration as practiced in the United States is described and the importance and limitations in evaluating urban stormwater based impacts are discussed.

Urban Stormwater Control. The use of storage facilities for retention, infiltration/percolation, and flow attenuation is outlined briefly and potential resulting water quality benefits are discussed. Optional supplemental or alternative controls to storage are described and the need for an integrated approach is stressed.

Section 3 - Terminology and Classification of Storage/Sedimentation Facilities

Terminology. The most commonly used terms are defined.

Classification. The classifications of storage and/or sedimentation facilities are discussed based on (1) major function, (2) location within the sewerage network, and (3) technical configuration.

Section 4 - System Planning, Design Procedures, and Integration

Planning. Planning concepts, methodologies, and tools common to all storage and/or sedimentation applications are introduced. The need for goal setting and realistic appraisals as forerunners to design are stressed.

Design Procedures. Design procedures for both combined sewer systems and separate storm sewer systems are described. The topics covered include problem identification, data needs, determination of pollutant loads, identification of pollutant removal objectives, control optimization, pollutant budget analysis, and operating strategy for design.

Integration. The role of storage/sedimentation facilities in an integrated stormwater management plan is discussed. The concurrent growth of stormwater control systems and urban areas is examined. Retrofit of existing flood control and drainage facilities to maximize pollution control is discussed. Examples of urban stormwater control are described to illustrate the several points.

Section 5 - Design of Retention Storage Facilities

Design Considerations. The principal factors affecting design are the size and locations of the facilities. Factors that influence facilities size are described and include volume, surface area, soil permeability, and infiltration rates. Considerations related to the location and siting of the facilities are also introduced.

Design Procedures. A step-by-step procedure is described for the planning and design of retention facilities. The procedure includes basin sizing requirements such as flow routing and pollutant removals, facilities site evaluation, final design factors, and solids control and removal facilities.

Performance. The removal efficiencies and treatment effectiveness of retention ponds are discussed. Constituents of concern are described and include organic compounds, bacteria, and viruses.

Operations. Operational problems with stormwater retention facilities are highlighted along with solutions to mitigate these problems. Special attention is given to the control and removal of solids and floatables.

Costs. Cost curves are presented for estimating the construction costs of retention and storage ponds.

Section 6 - Design of Detention Facilities

Design considerations are discussed for onsite detention and for in-system detention storage. Onsite detention is exemplified by rooftops, parking lots and streets, drainage swales, and detention reservoirs. In-system detention includes available storage in underground conduits and offline storage. A step-by-step design approach is presented for both onsite and in-system detention. Operation and maintenance considerations are also discussed.

Section 7 - Design of Storage/Sedimentation Facilities.

Discussions similar to Sections 5 and 6 are presented for the design of storage/sedimentation facilities. Representative process schematics are presented for planned or constructed facilities in the United States. Data are given on actual and expected removal rates for several facilities. A step-by-step design procedure is provided as well as preliminary cost data.

Section 8 - International Perspective.

A technology review of several international projects is presented. The review includes flow control devices developed in Sweden, Denmark, and Germany; a flow balancing system developed and applied in Sweden; and an innovative self-cleaning storage/sedimentation basin used in Zurich, Switzerland.

Appendixes

Appendix A - Pollutant Characterization and Estimation of Removal. The characterization of pollutants based on sample collection and analysis, pollutants to be analyzed, particle size determination, and pollutant distribution versus particle size and specific gravity are presented. A suggested analytical method for flow routing and pollutant routing is described.

Appendix B - Assessment Methods. Several desk top and computer hydrologic and stormwater pollution analysis methods are listed.

Appendix C - Infiltration Measurement Techniques. Soil infiltration rate testing procedures for use in investigating stormwater retention sites are presented.

SPECIAL ACKNOWLEDGMENTS

It has been stated earlier that recognition of urban stormwater runoff as a significant pollutant source and the development and application of countermeasures are receiving increasing attention from administrators, planners, and designers. This is particularly true in the area of storage and/or sedimentation controls where, concurrent with the development of this manual, three major projects, related in subject matter, have been initiated or completed.

Through the efforts of USEPA's Storm and Combined Sewer Section, and in particular Mr. Richard Field, Chief and Project Officer for this study, drafts

of portions of these documents have been made available to us. With the authors' permission, pertinent comments, data, and concepts have been used and are so referenced in this manual.

Because the time available to scan and incorporate highlights from this material has been very brief, interested readers are encouraged to seek out and review the following documents:

- Stahre, P. Flow Equalization in Sewer Systems. Report No. ISBN 91 540-3455 for the State Council for Building Research. Stockholm, Sweden. March 1981.
- Technical Wastewater Union E.V. (West Germany). Guidelines for the Sizing and Design of Stormwater Discharges in Combined Sewers. Working Instruction A-128. Draft. July 1977.
- American Public Works Association. Urban Stormwater Management - Special Report No. 49. 1981.

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1. American Public Works Association. Urban Stormwater Management - Special Report No. 49. 1981.

2. Armandes, C., and P. Bedient. Stormwater Detention in Developing Watersheds. Journal of Environmental Engineering Division, ASCE. April 1980.

3a. U.S. Environmental Protection Agency, Office of Wastewater Management. Combined Sewer Overflows -- Guidance for Long-Term Control Plan. USEPA Report No. EPA/832-B-95-002. September 1995.

3b. American Public Works Association. Nationwide Costs to Implement BMPs. 1992.

4. Lager, J. A. et al. Urban Stormwater Management and Technology: Update and Users' Guide. USEPA Report No. EPA/600/8-77-014. NTIS No. PB 275 654. September 1977.

Section 2

URBAN STORMWATER: AN OVERVIEW

Stormwater control and disposal have been recognized as serious urban problems for many centuries. Traditionally, the major goal of stormwater management has been to reduce the incidence and severity of flooding. Other goals include control of soil erosion and sedimentation, and protection and enhancement of stormwater as a groundwater recharge source. Increasingly, stormwater also is being recognized as a significant source of urban water pollution.

Hydrology is the study of the occurrence, distribution, movement, and properties of the water on the earth. Since there are numerous texts available that discuss hydrology in detail, a review of the hydrologic fundamentals will not be presented here. If the reader needs additional background in this area, a typical text is that of W. Viessman *et al*. [1]. However, in brief summary, the effects of urbanization on the way in which stormwater runoff is generated are to increase the rainfall excess portion of the hyetograph and to increase the peak of the hydrograph. Hydrographs and hyetographs for the same rain event on a watershed under both developed and undeveloped conditions are shown in Figure 3. As can be seen, the time to peak decreases and the runoff volume increases with urbanization.

The pollutants often found in urban runoff and CSOs are described and their importance assessed. Commonly applied stormwater control methods to minimize adverse impacts are introduced.

URBAN STORMWATER POLLUTION

As precipitation falls on and travels through the urban environment, it contacts and is contaminated by pollutants. Falling through the air, precipitation dissolves (e.g., "acid rain") and collects pollutants such as smog, dust, particulate matter, vapors, gases, etc. While typically the airborne contaminant pickup is minor [3], it may logically be assumed to be directly related to common air pollution indicies and thus is intensified downwind of stack and vehicular emissions.

Additional pollutants are dislodged and may be dissolved or suspended in the runoff as raindrops fall on the ground, structures, plants, chemical stockpiles, and littered surfaces. Thus, density of development, soil types, slopes and erodibility, and basic neighborhood cleanliness are logical further indicators of potential pollution. A diminishing source of available source pollutants due to washoff may account for reduced pollution loads from sequential storms or sequential periods within a single storm.

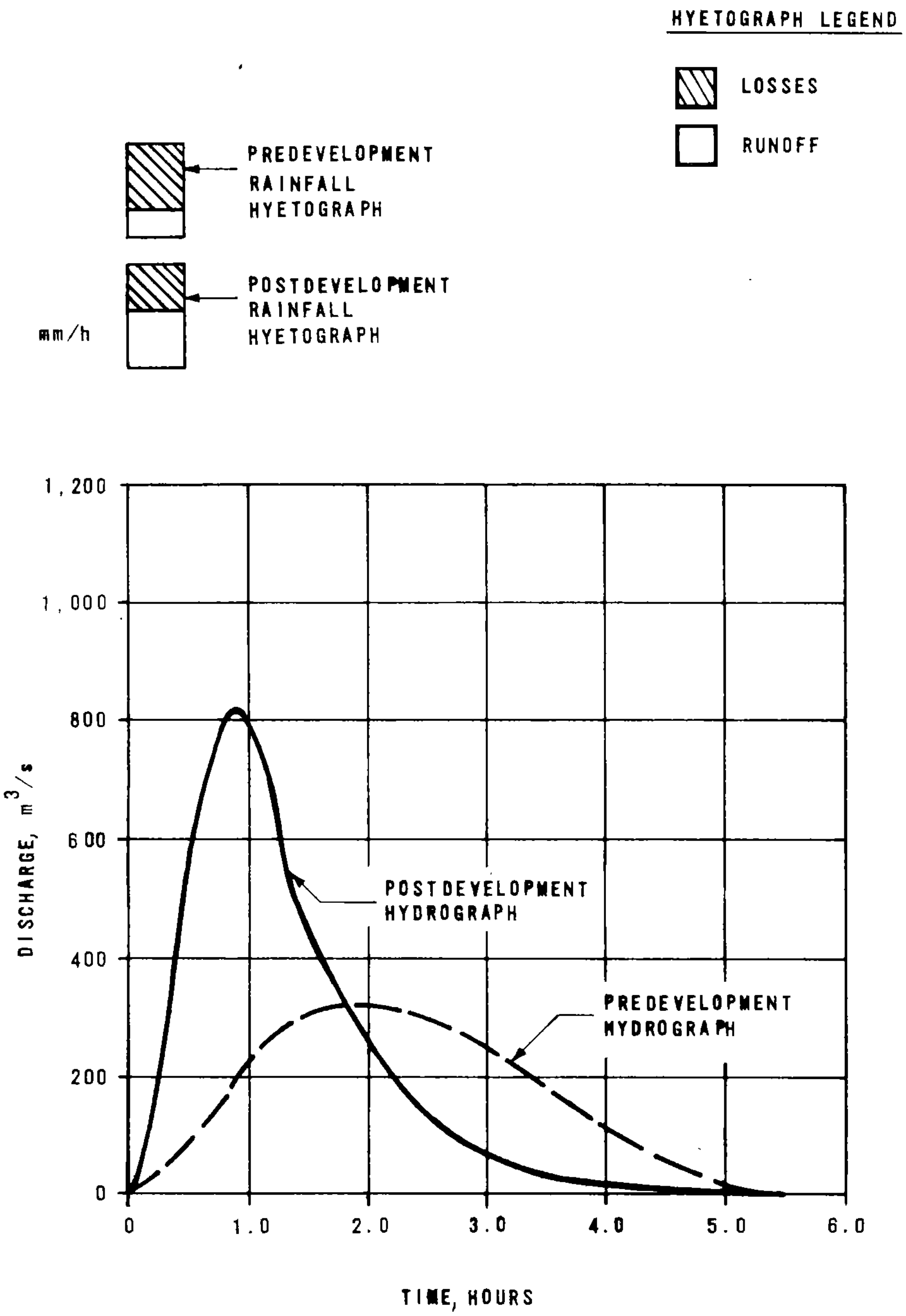

Figure 3. Hyetographs and hydrographs for watershed under predevelopment and postdevelopment conditions(adapted from [2]).

Finally, the collection system itself (whether open channels or closed conduits, separate or combined sewer systems) provides for additional contaminant contact opportunity from which the pollutant gain may be proportional to prestorm solids deposition (source material) and flow velocities and turbulance (resuspension ability). In combined sewer systems, obviously the sanitary and industrial wastewater is a major factor. In terms of oxygen demanding substances, University of Florida investigators [4] found "national average" CSO concentrations to be approximately four times the equivalent separate stormwater discharges. Similarly, bacterial contamination from untreated CSO discharges is typically two orders of magnitude higher than separate stormwater, although both are far above limits considered safe for body contact recreational use.

Representative Concentrations

Urban stormwater, whether conveyed in separate sanitary and storm sewer systems or combined sewer systems, essentially has all of the pollutants found in sanitary wastewaters. The concentration levels, however, vary markedly from location to location, storm to storm, and within a single storm. Representative mean values based on end-of-the-pipe samples from separate storm sewer and combined sewer systems are shown in Tables 1 and 2, respectively. The averages of these mean values are compared to representative uncontaminated background receiving water levels and "typical" raw sanitary wastewater in Table 3. The reader is cautioned that these values represent normalized values from a broad mix of observations and that site specific monitoring and sampling is essential for detailed goal setting and design.

Table 1. AVERAGE POLLUTANT CONCENTRATIONS IN STORMWATER RUNOFF [5]
mg/L Unless Otherwise Noted

City	TSS	VSS	BOD	COD	Kjeldahl nitrogen	Total nitrogen	Phosphorus	Ortho-phosphate	Lead	Fecal coliforms[a]
Atlanta, Georgia	287	--	9	48	0.57	0.82	0.33	--	0.15	6,300
Des Moines, Iowa	419	104	56	--	2.09	3.19	0.56	0.15	--	--
Durham, North Carolina	1,223	122	--	170	0.96	--	0.82	--	0.46	230
Knoxville, Tennessee	440	--	7	98	1.9	2.5	0.63	0.30	0.17	20,300
Oklahoma City, Oklahoma	147	--	22	116	2.08	3.22	1.00	1.00	0.24	40,000
Tulsa, Oklahoma	367	--	12	86	0.85	--	--	0.38	--	420
Santa Clara, California	284	70	20	147	--	5.8	0.23	--	0.75	--
Pullach, Germany	158	53	11	125	--	--	--	--	--	--
Average (not weighted)	415	88	20	113	1.41	3.11	0.62	0.46	0.35	13,500
Range	147-1,223	53-122	7-56	48-170	0.57-2.09	0.82-5.8	0.33-1.00	0.15-1.00	0.15-0.75	230-40,000

a. Organisms/100 mL.

Table 2. AVERAGE POLLUTANT CONCENTRATIONS IN COMBINED SEWER OVERFLOWS [5]
mg/L Unless Otherwise Noted

Location	TSS	VSS	BOD	COD	Kjeldahl nitrogen	Total nitrogen	PO_4-P	OPO_4-P	Lead	Fecal coliforms[a]
Des Moines, Iowa	413	117	64	--	--	4.3	1.86	1.31	--	--
Milwaukee, Wisconsin	321	109	59	264	4.9	6.3	1.23	0.86	--	--
New York City, New York										
Newton Creek	306	182	222	481	--	--	--	--	0.60	--
Spring Creek	347	--	111	358	--	16.6	4.5[b]	--	--	--
Poissy, France[c]	751	387	279	1,005	--	43	17[b]	--	--	--
Racine, Wisconsin	551	154	158	--	--	--	2.78	0.92	--	201,000
Rochester, New York	273	--	65	--	2.6	--	--	0.88	0.14	1,140,000
Average (not weighted)	370	140	115	367	3.8	9.1	1.95	1.00	0.37	670,000
Range	273-551	109-182	59-222	264-481	2.6-4.9	4.3-16.6	1.23-2.78	0.86-1.31	0.14-0.60	201,000-1,140,000

a. Organisms/100 mL.

b. Total P (not included in average).

c. Not included in average because of high strength of municipal wastewater when compared to the United States.

Table 3. COMPARISON OF STORMWATER DISCHARGES TO OTHER POLLUTANT SOURCES [5]
mg/L Unless Otherwise Noted

	TSS	VSS	BOD_5	COD	Kjeldahl nitrogen	Total nitrogen	Total PO_4-P	OPO_4-P	Lead	Fecal coliforms[a]
Background levels [6]	5-100	--	0.5-3	20	--	0.05-0.5[b]	0.01-0.2[c]	<0.01	<0.1	<1
Stormwater runoff	415	90	20	115	1.4	3.10	0.6	0.4	0.35	13,500
Combined sewer overflow	370	140	115	367	3.8	9.10	1.9	1.0	0.37	670,000
Sanitary wastewater	200	150	200	500	40	40	10	0.05-1.27	0.17	750,000

a. Organisms/100 mL.

b. NO_3 as N.

c. Total phosphorus as P.

Note: Background levels are the nonpoint pollution loads that can arise from land that has not been disturbed by man's activities.

For Tables 1-3: TSS - Total Suspended Solids; VSS - Volatile Suspended Solids; BOD - Biochemical Oxygen Demand; COD - Chemical Oxygen Demand

Concern over toxicity and trace substances in wastewater led USEPA to establish an extensive list of priority pollutants. As an aid to future investigators, laboratory results from nationally collected grab samples processed by USEPA's Region II laboratory are reported in Tables 4 and 5. The data represent the liquid fraction and sediment fraction analyses, respectively. The characteristically high copper, lead, and zinc values for highway runoff are apparent as well as a predominance of petroleum-derived hydrocarbons in all samples.

In combined systems and to a lesser extent in separate storm drains, many investigators report a characteristic pattern of high pollutant concentrations early in the period of storm runoff followed by diminishing concentrations as the storm progresses and presumably as the source of readily suspendable material and litter is washed away. Data that have been normalized to accentuate this "first flush" effect are shown in Figure 4.

Three factors appear to be of major significance in the degree to which a first flush is observed: (1) the quantity and location of readily suspended or resuspended material; (2) the intensity of the runoff (since suspension and resuspension are a function of flow velocity); and (3) the size of and time of travel in the collection system. If the collection system is large, a downstream observer will see a stormwater mixture of runoff components having vastly different travel times, thus dampening and extending concentration peaks. Similarly, the dependency on high intensity runoff may place the first flush anywhere in the storm according to the rainfall pattern, providing a similar scouring opportunity has not yet occurred.

Impacts of Urbanization

The tendency of urban development to intensify stormwater pollution was introduced earlier. The following example illustrates probable impacts on a macroscale.

Under a contract for the Association of Bay Area Governments [8], simplified mathematical model techniques coupled with a local monitoring program were used to address regional stormwater pollution over the nine county 13,000 km^2 (5,000 mi^2) area. Comparisons of annual pollutant loadings to San Francisco Bay under pre- and post-development conditions are shown in Table 6. Predevelopment loadings were simulated by setting runoff and pollution unit parameters to monitored present open-space values. (For example, in 1981, 87% of the total land area remained in a nominally undeveloped-open-space condition.)

Table 4. PRIORITY POLLUTANTS MEASURED IN URBAN STORMWATER SYSTEMS - LIQUID FRACTION
Concentrations in μg/L

	Combined wastewater				Stormwater runoff										
Parameter↓ Site/sample key→	A	B	C	D	E	F_1	F_2	G_1	G_2	H_1	H_2	I	J	K	L
Aromatics															
Benzene	0.15	--	0.59	--	--	--	--	--	--	--	--	--	0.28	0.46	--
Chlorobenzene	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--
1,3-Dichlorobenzene	5.50	--	--	--	--	--	--	--	--	--	--	--	--	--	--
1,4-Dichlorobenzene	5.50	4.70	--	--	--	--	--	--	--	--	--	--	--	--	--
Ethylbenzene	--	--	--	--	--	--	--	--	--	--	--	--	--	--	0.09
Toluene	3.90	2.00	3.10	1.30	1.10	--	--	--	--	--	--	--	--	7.70	4.20
1,2-Dichlorobenzene	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--
Polynuclear aromatic hydrocarbons															
1,2-Benzanthracene	--	--	--	--	--	--	--	50.00	--	--	--	3.40	--	--	--
3,4-Benzofluoranthene	--	--	--	--	--	--	--	51.50	--	--	--	8.60	--	--	--
11,12-Benzofluoranthene	--	--	--	--	--	--	--	51.50	--	--	--	8.60	--	--	--
Benzo(a)pyrene	--	--	--	--	--	--	--	170.00	--	--	--	--	--	--	--
Indeno(1,2,3-cd)pyrene	--	--	--	--	--	--	--	86.00	--	--	--	--	4.30	--	--
1,12-Benzoperylene	--	--	--	--	--	--	--	41.00	--	--	--	--	1.90	--	--
Acenaphthene	--	--	--	--	--	--	--	1.10	--	--	--	0.50	--	--	--
Acenaphthylene	--	--	--	--	--	--	--	0.22	--	--	--	0.20	--	--	--
Anthracene	0.39	--	--	--	--	--	--	26.50	--	--	--	0.60	--	--	--
Chrysene	--	--	--	--	--	--	--	50.00	--	--	--	3.40	--	--	--
Fluoranthene	--	--	0.30	--	--	--	0.30	98.00	--	1.10	0.40	12.00	--	--	0.03
Fluorene	--	--	--	--	--	--	--	1.00	--	--	--	0.60	--	--	--
Naphthalene	--	--	--	--	--	--	--	--	--	--	--	1.80	--	--	--
Phenanthrene	0.39	0.08	--	0.10	--	--	--	26.50	--	0.40	--	6.90	--	--	--
Pyrene	--	0.10	0.40	--	--	--	--	76.00	--	0.90	0.40	8.0	--	--	0.08
1,2:5,6-Dibenzanthracene	--	--	--	--	--	--	--	--	--	--	--	--	7.20	--	--
Halogenated hydrocarbons															
Tetrachloroethylene	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--
Methyl chloride	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--
Methylene chloride	0.70	--	--	--	--	--	--	--	4.60	1.40	--	--	0.95	--	--
Chloroform	4.80	3.60	0.51	0.10	1.10	--	--	--	--	--	--	--	--	--	--
Hexachloroethane	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--
1,2-Dichloropropane	--	--	--	--	--	--	--	--	--	--	--	--	--	--	0.10
Dichlorobromomethane	--	--	--	--	0.10	--	--	--	--	--	--	--	--	--	--
Trichlorofluoromethane	--	--	--	--	1.90	--	--	--	--	--	--	0.20	2.90	--	--
1,2-Trans-dichloroethylene	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--
1,1,1-Trichloroethane	--	--	--	--	--	--	--	--	--	--	--	--	--	2.10	--
Trichloroethylene	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--
Phthalates															
Diethyl phthalate	7.0	0.30	--	1.50	0.50	0.50	--	--	0.80	--	--	0.30	0.35	--	0.06
Di-n-butyl phthalate	4.60	0.40	--	3.80	1.50	3.40	2.20	1.90	1.80	0.50	0.70	2.8	--	--	0.30
Bis(2-ethylhexyl) phthalate	110.0	1.60	2.10	7.60	2.70	32.0	7.70	160.0	7.70	3.40	6.20	19.0	2.30	--	2.60
Butylbenzyl phthalate	4.80	--	--	--	0.50	--	--	--	--	--	--	5.0	--	--	--
Di-n-octyl phthalate	3.60	--	--	--	--	--	--	--	--	--	--	--	--	--	--

Table 4 (Concluded)

Parameter↓ Site/sample key→	Combined wastewater				Stormwater runoff										
	A	B	C	D	E	F_1	F_2	G_1	G_2	H_1	H_2	I	J	K	L
<u>Phenols</u>															
Phenol	18.0	--	0.32	--	0.70	0.30	0.70	0.34	0.85	--	0.40	1.90	--	--	--
Pentachlorophenol	--	--	--	--	0.90	0.90	0.90	--	--	--	--	--	--	--	--
Phenolics	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--
2,4-Dimethylphenol	--	--	--	--	--	0.50	0.70	--	--	--	--	--	--	--	--
2,4,6-Trichlorophenol	--	--	--	--	--	0.30	--	--	--	--	--	--	--	--	--
<u>Nitroso compounds</u>															
N-Nitrosodiphenylamine	--	--	--	--	--	--	--	--	--	--	--	0.80	--	--	--
<u>Pesticides</u>															
4,4'-DDE	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--
<u>Metals</u>															
Antimony	<60.0	<20.0	<20.0	<20.0	<40.0	<20.0	<20.0	<20.0	<20.0	<20.0	<20.0	<20.0	<60.0	--	<20.0
Arsenic	~0.60	<1.0	~3.0	<0.4	<0.20	~5.0	~4.0	<2.0	<2.0	15.0	~3.0	~1.0	~2.0	--	<0.4
Beryllium	<4.0	<1.0	<1.0	<1.0	<3.0	<2.0	<2.0	<1.0	<1.0	<1.0	<1.0	<3.0	<4.0	--	<1.0
Cadmium	<3.0	<3.0	~2.0	<3.0	<2.0	<4.0	~7.0	~7.0	~5.0	~5.0	~3.0	9.8	<3.0	--	<3.0
Chromium	<10.0	<7.0	~10.0	<7.0	<7.0	<9.0	<9.0	~20.0	~10.0	~20.0	36.0	43.0	<10.0	--	<7.0
Copper	110.0	<2.0	40.0	<2.0	14.0	~40.0	<40.0	24.0	11.0	47.0	49.0	280.0	~20.0	--	<2.0
Lead	~100.0	<30.0	~30.0	<30.0	~100.0	107.0	113	360.0	310.0	~80.0	400.0	2600.0	~70.0	--	<30.0
Mercury	<0.2	<0.2	0.25	<0.20	<0.2	0.45	0.30	<0.20	0.56	<0.2	<0.2	<0.20	0.67	--	0.30
Nickel	~20.0	<9.0	<9.0	<9.0	<9.0	<40.0	<40.0	<9.0	<9.0	<30.0	<30.0	~40.0	<10.0	--	<9.0
Selenium	<0.70	<0.8	<10.0	~1.0	~1.0	<3.0	<3.0	<10.0	<10.0	<4.0	<4.0	<0.80	~0.80	--	<0.8
Silver	<10.0	<3.0	<3.0	<3.0	<6.0	<8.0	<8.0	<3.0	~5.0	<7.0	<7.0	<2.0	<10.0	--	<3.0
Thallium	<0.40	<0.4	<10.0	<0.40	<0.2	<2.0	<2.0	<10.0	<10.0	<4.0	<4.0	<0.40	<0.40	--	<0.40
Zinc	210.0	20.0	~100.0	~10.0	65.0	140.0	~90.0	330.0	~90.0	170.0	260.0	780.0	120.0	--	23.0

Notes:

1. A - Boston, Massachusetts; B - Clatskanie, Oregon; C - Rochester, New York; D - St. Helens, Oregon; E - Austin, Texas; F_1, F_2 - San Jose, California (Coyote Creek): G_1, G_2 - Orlando, Florida (Lake Eola); H_1, H_2 - Trenton, New Jersey (Lawrence Shopping Center); I - Milwaukee, Wisconsin (Interstate 94); J - New Roxbury, Massachusetts; K - Rochester, New York; L - St. Helens, Oregon.
2. Samples were predominantly grab samples, collected by different observers in 1979-80, and processed on a time available basis, at the USEPA, Region II Surveillance and Analysis Division Laboratory, in Edison, New Jersey, under the supervision of the USEPA Project Officer, Richard Field, Chief, Storm and Combined Sewer Section, Municipal Environmental Research Laboratory - Cincinnati in Edison, New Jersey [7].

Table 5. PRIORITY POLLUTANT MEASURED IN URBAN STORMWATER SYSTEMS - SEDIMENT FRACTION
Concentrations in μg/kg (except as noted)

Parameter↓ Site/sample key→	Combined sewers A	B	C	D	E_1	E_2	Storm drains or outlets F	G	H
Aromatics									
Benzene	--	--	2.0	--	--	2,000.0	--	--	--
Chlorobenzene	--	--	--	--	0.60	--	--	--	--
1,3-Dichlorobenzene	--	310.0	--	560.0	73.0	--	--	--	--
1,4-Dichlorobenzene	--	310.0	--	560.0	73.0	--	--	--	--
Ethylbenzene	17.0	20.0	--	8.60	1.10	1,100.0	12.0	--	--
Toluene	5,900.0	910.0	42.0	32,000.0	94.0	1,300.0	100.0	--	0.20
1,2-Dichlorobenzene	2,300.0	--	--	--	--	150.0	--	--	--
Polynuclear aromatic hydrocarbons									
1,2-Benzanthracene	110,000.0	1,100.0	4,100.0	1,600.0	19,000.0	16,000.0	2,400.0	--	1,100.0
3,4-Benzofluoranthene	56,000.0	530.0	2,300.0	870.0	13,000.0	12,000.0	1,200.0	--	1,400.0
11,12-Benzofluoranthene	56,000.0	530.0	2,300.0	870.0	13,000.0	12,000.0	1,200.0	--	1,400.0
Benzo(a)pyrene	75,000.0	∿2,800.0	25,000.0	∿4,500.0	20,000.0	12,000.0	1,200.0	--	600.0
Ideno(1,2,3-cd)pyrene	40,000.0	--	740.0	--	3,900.0	4,600.0	260.0	--	--
1,12-Benzoperylene	26,000.0	--	810.0	--	5,800.0	4,200.0	560.0	--	--
Acenaphthene	11,000.0	--	270.0	--	440.0	2,400.0	110.0	--	--
Acenaphthylene	--	--	--	--	1,100.0	--	--	--	--
Anthracene	140,000.0	720.0	7,000.0	930.0	10,000.0	20,000.0	1,100.0	375.0	430.0
Chrysene	110,000.0	1,100.0	4,100.0	1,600.0	19,000.0	16,000.0	2,400.0	--	1,500.0
Fluoranthene	110,000.0	1,000.0	4,200.0	1,400.0	15,000.0	18,000.0	2,000.0	1,200.0	2,300.0
Fluorene	11,000.0	--	300.0	--	1,100.0	2,200.0	87.0	--	210.0
Naphthalene	4,900.0	--	--	--	170.0	900.0	47.0	--	--
Phenanthrene	140,000.0	720.0	--	930.0	10,000.0	20,000.0	1,100.0	375.0	2,000.0
Pyrene	81,000.0	800.0	3,000.0	1,200.0	13,000.0	13,000.0	1,600.0	1,200.0	2,600.0
1,2:5,6-Dibenzanthracene	--	--	--	--	--	--	--	--	--
Halogenated hydrocarbons									
Tetrachloroethylene	--	--	--	--	--	59,000.0	--	--	--
Methyl chloride	--	--	--	--	--	--	0.20	--	--
Methylene chloride	--	--	--	--	--	--	1.70	--	1.30
Chloroform	16.0	--	--	0.20	--	1,700.0	--	--	--
Hexachloroethane	--	--	1,500.0	--	--	--	5,600.0	--	--
1,2-Dichloropropane	--	--	--	--	--	--	--	--	--
Dichlorobromomethane	--	--	--	--	--	--	--	--	--
Trichlorofluoromethane	--	--	--	--	--	--	--	--	--
1,2-Trans-dichloroethylene	--	0.70	--	0.60	--	43,000.0	--	--	--
1,1,1-Trichloroethane	--	--	--	--	--	--	--	--	--
Trichloroethylene	--	--	--	--	--	160,000.0	--	--	--
Phthalates									
Diethyl phthalate	--	--	--	--	--	--	--	--	--
Di-n-butyl phthalate	2,000.0	1,300.0	300.0	850.0	160.0	840.0	860.0	2,500.0	370.0
Bis(2-ethylhexyl) phthalate	7,700.0	2,600.0	1,100.0	3,800.0	360.0	40,000.0	1,800.0	59,000.0	130.0
Butylbenzyl phthalate	2,300.0	190.0	450.0	320.0	--	--	--	1,200.0	--
Di-n-octyl phthalate	--	--	--	--	--	--	--	--	--

Table 5 (Concluded)

Parameter↓ Site/sample key→	Combined sewers A	B	C	D	E_1	E_2	Storm drains or outlets F	G	H
Phenols									
Phenol	--	~300.0	140.0	~60.0	--	--	32.0	--	--
Pentachlorophenol	--	--	--	--	--	--	--	--	--
Phenolics	--	--	--	--	--	--	--	--	--
2,4-Dimethylphenol	--	--	--	--	--	--	--	--	--
2,4,6-Trichlorophenol	--	--	--	--	--	--	--	--	--
Nitroso compounds									
N-Nitrosodiphenylamine	--	--	--	--	--	--	--	--	--
Pesticides									
4,4'-DDE	--	--	--	--	--	--	180.0	--	--
Metals, mg/kg									
Antimony	<2.0	<3.0	3.30	~8.0	12.0	~6.0	16.0	--	<3.0
Arsenic	1.70	2.80	4.80	3.50	3.30	2.90	2.30	--	2.80
Beryllium	<1.0	<0.20	~0.10	<0.2	~0.10	~0.30	<0.20	--	<0.20
Cadmium	~2.0	2.30	<0.06	2.80	0.50	5.5	~0.80	--	~1.0
Chromium	32.0	26.0	28.0	25.0	42.0	110.0	14.0	--	8.10
Copper	920.0	200.0	140.0	1,000.0	83.0	1,000.0	17.0	--	7.0
Lead	560.0	160.0	370.0	360.0	410.0	37.0	1,300.0		24.0
Mercury	0.63	0.78	2.10	0.74	0.32	13.0	0.26	--	2.39
Nickel	18.0	13.0	15.0	23.0	19.0	67.0	5.90	--	~3.0
Selenium	<0.02	0.20	0.06	0.12	0.27	<0.02	2.80	--	1.40
Silver	<0.40	<0.50	2.10	8.90	6.10	~1.0	<0.50	--	<0.5
Thallium	~0.01	<0.20	<0.20	<0.20	<0.20	<0.20	<0.20	--	<0.20
Zinc	700.0	910.0	380.0	710.0	120.0	300.0	160.0		37.0

Notes:

1. A - Boston, Massachusetts; B - Clatskanie, Oregon; C - Rochester, New York; D - St. Helens, Oregon; E_1, E_2 - Syracuse, New York; F - Austin, Texas; G - San Jose, California (Coyote Creek); H - Trenton, New Jersey (Lawrence Shopping Center).
2. Samples were predominantly grab samples, collected by different observers in 1979-80, and processed on a time available basis, at the USEPA, Region II Surveillance and Analysis Division Laboratory, in Edison, New Jersey, under the supervision of the USEPA Project Officer, Richard Field, Chief, Storm and Combined Sewer Section, Municipal Environmental Research Laboratory - Cincinnati in Edison, New Jersey [7].

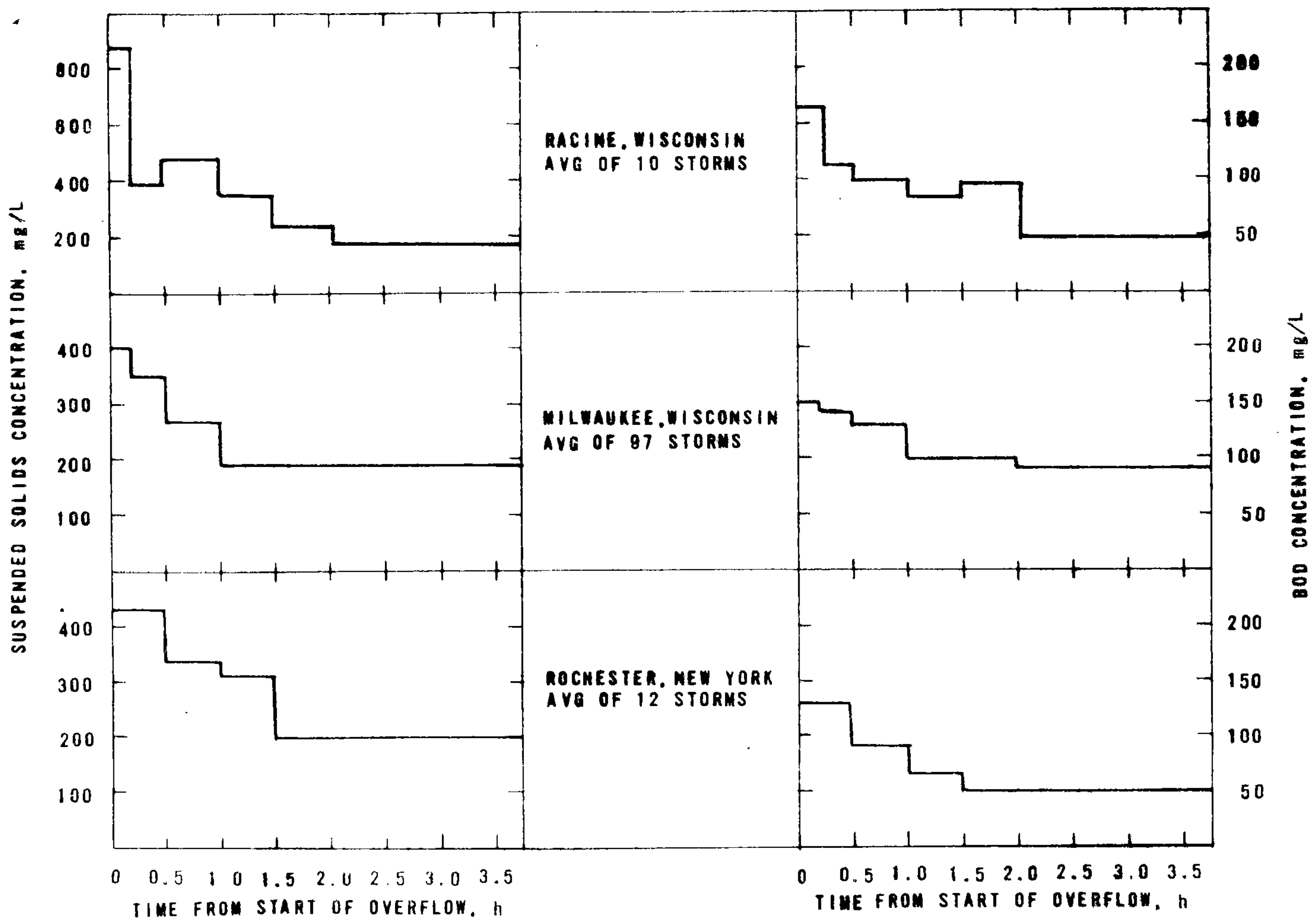

Figure 4. Combined sewer overflow quality versus time for selected cities [5].

Table 6. SURFACE RUNOFF LOADINGS TO BAY, PRE- AND POST-DEVELOPMENT [8]

Parameter	Average annual loading, 1000s lb[a] 1800	1975	Increase
Total BOD	3,190	18,590	5.8:1
Total SS	790,091	765,982	0.97:1
Total nitrogen	1,193	3,677	3.1:1
Total phosphorus	89	428	4.8:1

a. Based on 1969-1970 and 1970-1971 water year rainfall data.

lb x 0.4536 = kg

The following observations were made:

> It can be seen that organic loadings have probably increased about 6-fold and nutrients about 4- to 5-fold while total solids have remained relatively unchanged. Thus it appears that the Bay, as in most shallow embankments, has always been turbid and no actions by man will likely reverse this condition. With respect to dissolved oxygen, the combined actions of Bay filling and increased organics discharged have, no doubt, worsened conditions in backwaters and areas of poor circulation. While nutrient loadings have grown substantially, their impact when compared to point discharged remains small. [8]

In terms of BOD loadings, it was further observed that while the nine county average change was an increase of 5.8:1, the range between counties was great--varying from 124:1 for the fully developed and combined sewered San Francisco to 1.7:1 for the predominantly rural and agricultural Sonoma County.

Receiving Water Assessments

The approach to receiving water protection and restoration in the United States over the past decade has entailed three principal activities: (1) a definition of beneficial uses and evaluation of the receiving waters assimilative capacity with respect to preserving or restoring these uses; (2) a systemized waste load allocation between competing point and nonpoint discharges within the limits of this calculated assimilative capacity; and (3) a national permitting program for dischargers to provide a vehicle for enforcement and compliance monitoring. The process was simplified by adoption of the 1972 Clean Water Act which set an effluent-based standard of secondary treatment for all publicly owned treatment works. In the case of separate stormwater and CSO treatment facilities, latitude was provided to permit a case-by-case approach at the discretion of USEPA Regional Administrators.

Field and Turkeltaub [9] in an overview of receiving water impacts of stormwater and CSO discharges conclude that "under certain conditions storm runoff can govern the quality of receiving waters regardless of the level of dry-weather flow treatment provided." Examples cited include Milwaukee, Wisconsin; Syracuse, New York; Seattle, Washington; Orlando, Florida; San Jose, California; and New York City, New York. Principal pollutants and impacts include bacterial contamination, toxic substances, sediment loads, and oxygen demand and depletion.

Heaney, et al. [10] summarized data from 248 urbanized areas in an attempt to quantify receiving water impacts from urban stormwater. The present data base was found to be poor; numerous definitions of "problems" are being used; and "...relatively little substantive data to document impacts have been collected." Impacts found were most noticeable in small streams, but impacts were difficult to isolate from other sources such as municipal and industrial wastes. Accidental or deliberate discharges from point sources under wet-weather conditions were sometimes the primary cause of wet-weather impacts.

URBAN STORMWATER CONTROL

Control of urban stormwater has been practiced for a number of centuries. The cloaca maxima, a storm drain constructed for the ancient Roman Forum, is still in use today. The primary goal of urban stormwater control has been to reduce the incidence and severity of flooding. In addition to direct conveyance facilities, the approach to flood control has often been to attenuate, or slow, the flow of runoff so that the peak rate of flow is reduced and the urban runoff hydrograph is made to more closely approximate the predevelopment hydrograph. The urban hydrograph from Figure 3 is reproduced in Figure 5, along with a storage facility attenuated hydrograph for the same watershed and storm.

Flow attenuation may be accomplished by increasing the permeability of the sides and bottom of unlined channels and basins, increasing the roughness of the flow surfaces, or by constructing storage facilities in which the runoff of CSO is temporarily stored and released slowly. Where earthen storage facilities take advantage of the permeability of the sides and bottom, the runoff volumes are reduced. The design of storage facilities where soil permeability is a major factor is described in Section 5. The design of detention storage facilities is described in Section 6. Sedimentation facility design is discussed in Section 7.

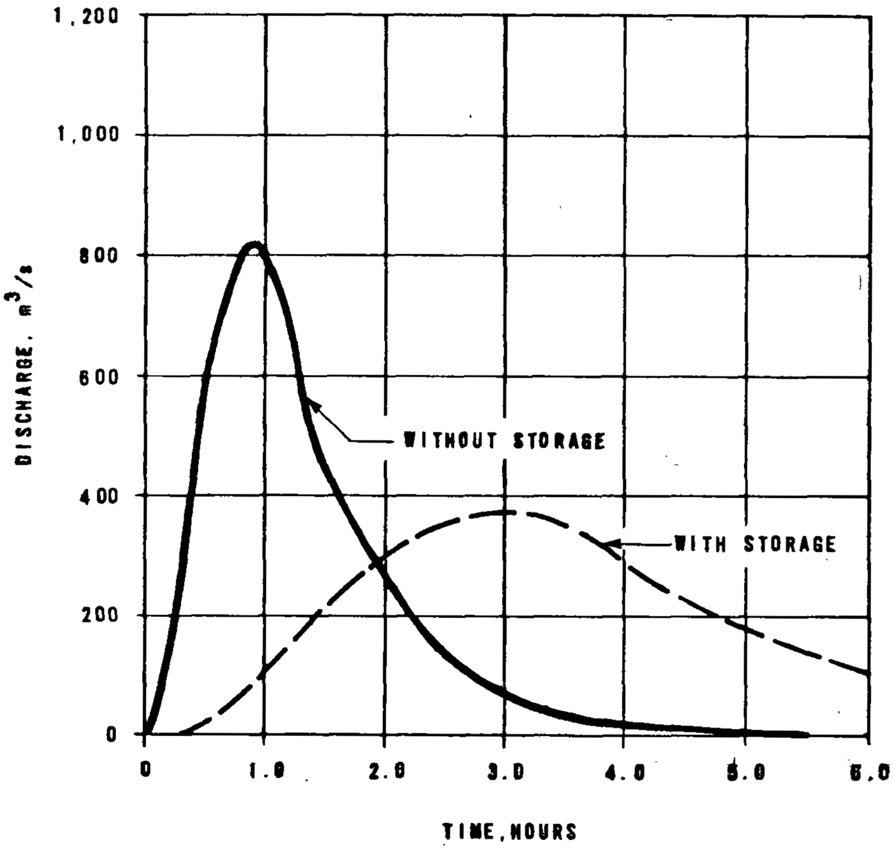

Figure 5. Hydrograph of a watershed showing effects of storage.

Water quality benefits may be realized from storage options in many ways, principally:

- Increased effectiveness of existing treatment facilities through flow equalization
- Increased interception of pollutant-laden runoff flows with subsequent diversion to treatment
- Reduced erosion and scour by flowrate control
- Groundwater recharge through increased infiltration/percolation opportunity
- Treatment by sedimentation during storage

- Treatment by biological stabilization in cases of extended, several days to several weeks, storage

- Reduced wastewater collection/treatment system overloads and bypasses when utilized in an integrated manner

Storage application should be approached in a systemized manner, such as through the classification system recommended in Section 3. Virtually every urban stormwater control will use storage to some degree; however storage by itself may not offer the most cost-effective solution.

Optimal supplemental controls to storage include best management practices, high-rate unit processes, and innovative systems approaches that optimize treatment and controls on a total systems basis. Extensive discussion of these options as applied to separate urban stormwater and CSO systems is available in the literature [5, 11, 12].

Best management practices (nonstructural and low structurally intensive alternatives) offer considerable promise as the first line of action to control urban runoff pollution. By treating the problem at its source, or through appropriate legislation curtailing its opportunity to develop, multiple benefits can be derived. These include lower cost, earlier results, erosion/flood control benefits, and an improved and cleaner neighborhood environment.

Physical treatment alternatives are primarily applied for solids removal from waste streams, and are of particular importance to CSO treatment for removal of settleable and suspended solids and floatable material. Physical treatment systems have a demonstrated capability to handle high and variable influent concentrations and flowrates and operate independently of other treatment facilities, with the exception of treatment and disposal of the sludge/solids residuals. The principal disadvantage relates to those periods of time when equipment sits idle during periods of dry weather. When implemented on a dual-use basis as either pretreatment or effluent polishing of conventional treatment plant flows, reduced capital investments may be realized.

The size and complexity of urban runoff management programs are such that there is a need for an integrated approach to their solution. The solution is most often a combination of various best management practices and unit process applications. Demonstrated implementation progress to date is predominately in the areas of CSO control, excess flow treatment from heavily infiltrated sanitary systems, and best management practices applications in new communities.

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Section 3

TERMINOLOGY AND CLASSIFICATION OF STORAGE AND/OR SEDIMENTATION FACILITIES

The term "Storage and/or Sedimentation Facility" is used here as a collective term for various arrangements for balancing, detaining, and storing the flow of stomwater (with or without any sedimentation) in combined sewer systems or the storm drainage portion of separate sewer systems.

For ease of reference and consistency, certain terminology and classifications of control systems have been adopted for use herein. The basic categories for classification of storage and/or sedimentation facilities are:

1. The main function principles that are applicable to both separate sanitary sewer and storm drainage systems and to combined sewer systems:
   - Retention
   - Detention
   - Sedimentation
2. The general placement of the storage and/or sedimentation facility or practice within the collection network:
   - Source controls (including onsite and upstream facilities)
   - In-system controls (storage developed within the transport network by regulators, flow restrictors, etc.; and storage developed in offline basins located along the transport network)
   - Downstream (end-of-pipe) controls
3. The technical configuration of the various facilities based on location, topography, and local conditions.

TERMINOLOGY

Various storage and/or sedimentation systems have been described in the technical literature in this field. To avoid any unnecessary confusion or misunderstandings, the terms most commonly used in this manual are defined as follows:

Combined Sewer--A sewer designed to receive both stormwater runoff and municipal sewage (sanitary and industrial wastewater).

Combined Sewer Overflow--The untreated discharge of a combined sewer system that generally occurs when the dry-weather flow treatment or interceptor capacity is exceeded.

Combined Sewer System--A system wherein storm drainage is commingled and conveyed with municipal sewage.

Detention Storage--The form of storage where stormwater runoff or CSO are stored temporarily. Stored flows are subsequently returned to the sewerage system at a reduced rate of flow when downstream capacity is available, or the flows are discharged to the receiving water with or without further treatment.

Downstream Control Options--Online or offline storage and/or sedimentation facility located immediately upstream of the receiving water or final treatment facility.

Downstream Storage--A storage facility designed to serve as a primary sedimentation treatment device and/or as a flow equalization device immediately upstream of a treatment works or receiving water.

Dry Ponds--Ponds which are normally dry and fill in response to storm conditions. The usable storage volume may include, where measureable, pore space in the basin walls and floor which are inundated rapidly in the filling process.

Inlet Control--Detention storage where stormwater is temporarily stored on impervious surfaces before it enters the sewer system.

Inline Storage--In-system control storage facilities, either in series or parallel with the collection system, which are filled and emptied only by gravity.

In-System Controls--Options wherein basins, tunnels, or caves in the collection network or temporary excess pipe capacity in the collection network are used for storage through the use of regulator devices (including both static and mechanical regulators) and pumping facilities.

Offline Storage--In-system control storage facilities where pumping is required to convey flow to the storage facility or to return the stored water to the sewer system. An advantage to offline storage is the easier flexibility to select when the stored flow is to be returned to the sewer for treatment or discharge.

Onsite Storage--Storage of stormwater in natural ditches, open ponds or basins, rooftops, parking lots, or recreational facilities (athletic fields, tennis courts, etc.) before the stormwater reaches a sewer network.

Retention Storage--Storage where stormwater runoff is captured and disposed of through infiltration, percolation, and evaporation. An emergency overflow structure must be included to prevent structural damage to the facility by occasional overflows.

Sanitary Sewer--A sewer that carries liquid and water-carried wastes from residences, commercial buildings, industrial plants, and institutions, together with relatively low quantities of groundwater, stormwater, and surface waters that are not admitted intentionally.

Separate Sewer System--A system wherein stormwater runoff is conveyed independent of sanitary and industrial wastewater. In reality, most separate stormwater systems contain at least a few cross-connections for relief of overloaded sanitary sewers.

Source Control Options--Facilities and practices, which initiate corrective action close to the origin of the stormwater runoff, to mitigate stormwater or CSO adverse impacts.

Storm Sewer--A sewer that carries stormwater runoff, street wash and other washwaters, or drainage, but excludes domestic sewage and industrial wastes.

Stormwater Runoff--Water resulting from precipitation which runs off freely from the surface, or is captured by storm sewer, combined sewer, or to a limited degree sanitary sewer facilities.

Wet Ponds--Ponds which are normally partly filled and in which storage is attained by a change in water surface elevation.

CLASSIFICATION

The classification or systemization of storage and/or treatment facilities can be based on the technical configuration (i.e., area available, topography, adjacent land use, local conditions, etc.); location within the sewerage network (i.e., prior to flow entering a pipe network, within the pipe network, or just upstream of a treatment plant or the receiving water); or function (i.e., retention, detention, or sedimentation).

The classification scheme and basic terminology used in this manual and shown in Figure 6 are based on location within the sewer network. However, the relationship of the classification and application of the various control options to separate storm sewer and combined sewer systems is shown in Figure 7. In current practice, source control retention and detention storage applies to separate storm sewer systems and to combined sewer systems upstream of the introduction of sanitary or industrial wastewater. To date, in-system and downstream controls have been applied mainly on combined sewer systems. An example of an in-system or downstream control on a storm sewer would be a basin, either inline or offline, used for flood control. Such a basin can be designed to take advantage of the sedimentation that occurs in a storage facility to improve the quality of the discharged flow while controlling the discharge rate.

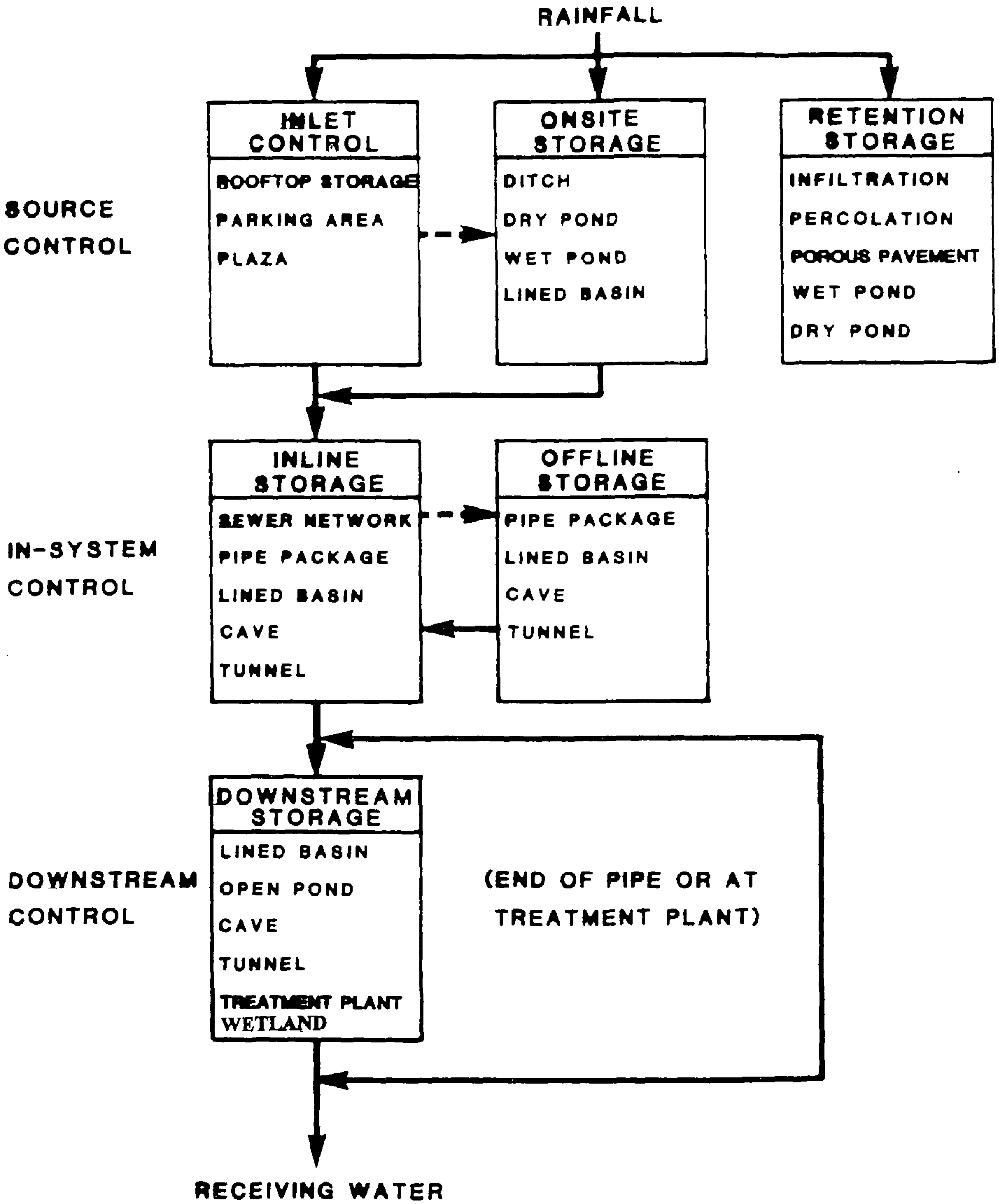
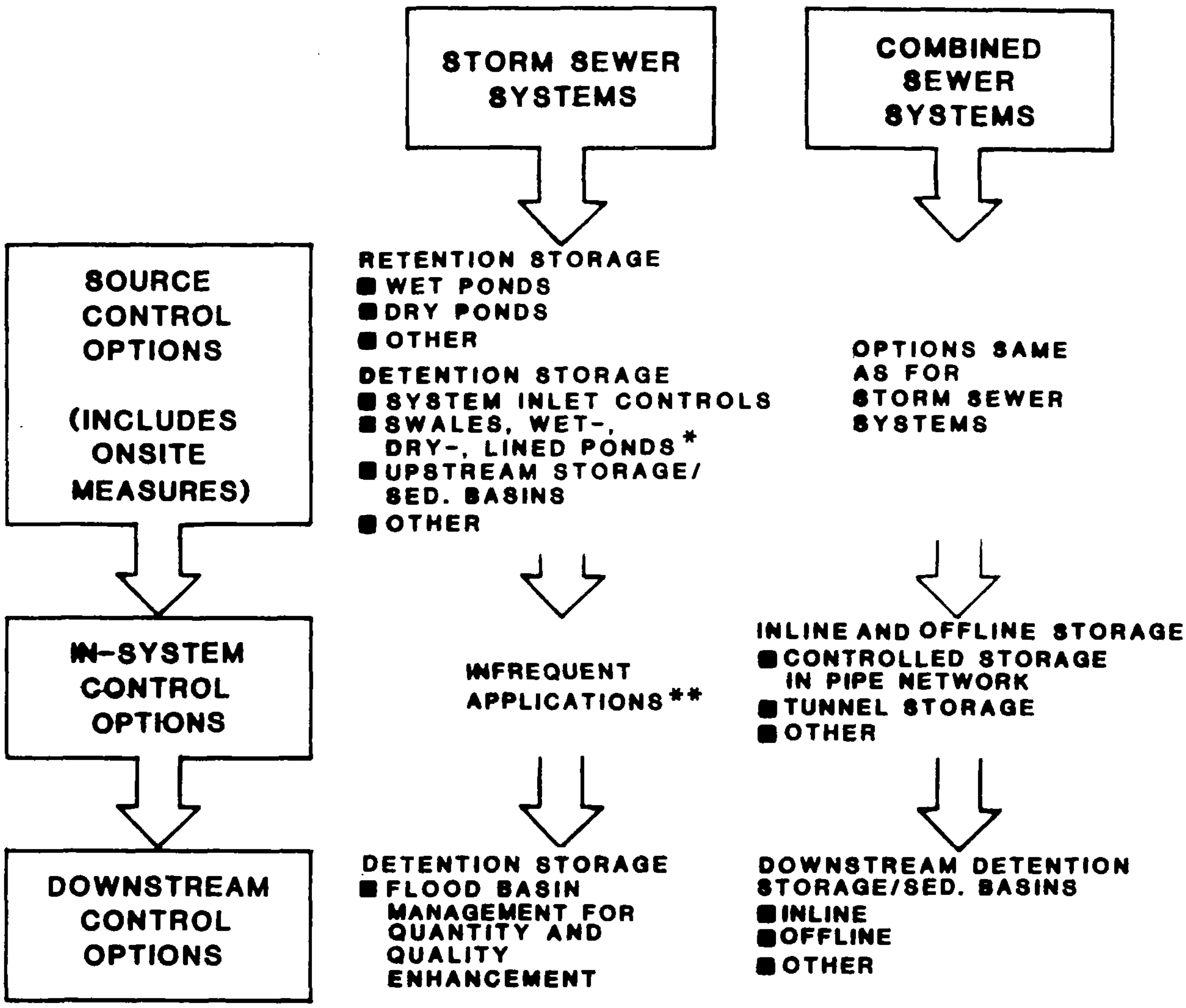


Figure 6. Schematic classification of storage and/or sedimentation facilities.



*DESIGNED FOR LIMITED FLOW CAPTURE VERSUS ESSENTIALLY FULL FLOW CAPTURE FOR RETENTION STORAGE ALTERNATIVES.

**WHERE NECESSITATED BY WATER QUALITY CONSIDERATIONS, TREATMENT (AS IN CSO OPTIONS) SHOULD BE EVALUATED (I.E., INTERCEPTION AND DIVERSION OF DRY-WEATHER SPILLS TO TREATMENT).

Figure 7. Classification scheme based on type of sewer system.

The relationship of the main grouping, technical configuration, and main function principle is shown in Table 7. A schematic of the classification of storage and/or sedimentation facilities according to their basic function of principle is shown in Figure 8.

Table 7. CLASSIFICATION OF STORAGE AND/OR SEDIMENTATION FACILITIES

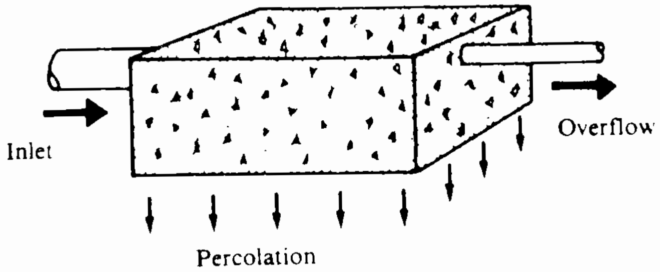
Main group	Technical configuration	Main function principle		
		Retention	Detention	Sedimentation
Source control	Rooftop	-	X	-
	Parking lot	-	X	-
	Plaza	-	X	-
	Ditch	X	X	-
	Dry pond	X	X	X
	Wet pond	X	X	X
	Lined basin	-	X	X
	Porous pavement	X	-	-
In-system control	Sewer network	-	X	-
	Pipe package	-	X	-
	Lined basin	-	X	-
	Cave	-	X	-
	Tunnel	-	X	-
Downstream control	Lined basin	-	X	X
	Open pond	-	X	X
	Cave	-	X	X
	Tunnel	-	X	X
	Treatment plant	-	X	X

There are three main function principles associated with storage and/or sedimentation facilities: (1) retention storage, (2) detention storage, and (3) sedimentation. Any storage and/or sedimentation facility can be designed primarily to meet any one of the main function principles (retention and sedimentation or detention and sedimentation).

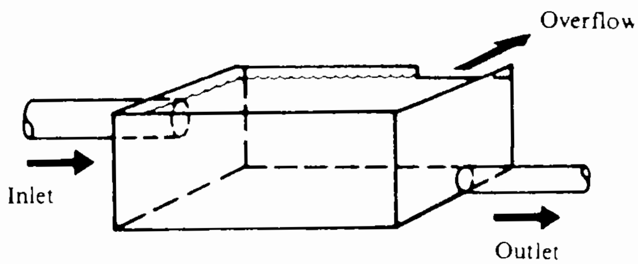
The main groupings of storage and/or sedimentation facilities by location within the sewerage network are: (1) source control, (2) in-system control, and (3) downstream control. Within these main groupings a variety of technical configurations can be accommodated.

Many of the technical configurations can be used to satisfy one or more main function principles or main groupings. The technical configurations include:

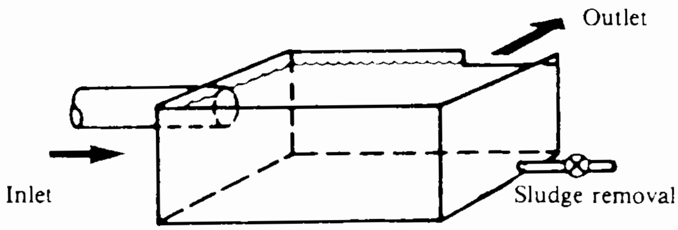
- Percolation basins
- Drainage swales
- Dry wells
- Trenches
- Porous pavement
- Blue-green storage
- Rooftop ponding
- Parking lots



a) Retention basin



b) Detention basin



c) Sedimentation basin

Figure 8. Classification of storage and/or sedimentation facilities based on main function of principle (adapted from [1]).

- Pedestrian plazas and malls
- Dry ponds
- Wet ponds
- Check dams
- In-pipe storage in existing sewers
- Pipe bundles
- Concrete basins
- Tunnels and caverns

Main Function Principles

Retention Storage. Retention is the storage of stormwater runoff for complete removal from the surface drainage and discharge system. The primary intent in the use of retention storage is to allow the stormwater to evaporate and/or to infiltrate and percolate into the ground.

Percolation of stormwater to the groundwater offers a number of benefits in addition to controlling stormwater flows. The groundwater is recharged helping to reduce or prevent ground subsidence; lowering the water table increases the overburden pressure of the soil located between the original and the lowered water table causing compression of the soil mass. This is particularly important in areas where the groundwater basins are being overdrawn and increased urbanization is reducing normal infiltration. Percolation of stormwater can be used as water supply recharge in areas where ground subsidence due to overdraft pumping is not a problem. In addition, percolation through a soil column has been shown to be very effective in removing bacteria, oxygen demanding material, and suspended material from wastewater.

Detention Storage. Detention is the storage of stormwater runoff for delaying and controlling the release rate to attenuate peak flows in the surface drainage and discharge system. Detention storage can be accomplished through the use of system inlet controls, online and offline ponds, or onsite storage.

Onsite detention refers to the storage of stormwater runoff at or very near the site of its origin, and its subsequent discharge at a predetermined release rate. The intent of onsite detention is to utilize existing or proposed impervious areas or structures to store and control runoff. Typical examples of such onsite storage include rooftops, plazas, parking lots and streets, underground structures, ponds, and multipurpose detention reservoirs.

Sedimentation. Sedimentation, as used here, refers to those facilities whose primary purpose is to separate suspended particles from water by gravitational settling. Sedimentation basins may also be referred to as sedimentation tanks, settling basins, or settling tanks.

The objective of treatment by sedimentation is to remove readily settleable solids and floating material and thus to reduce the suspended solids content. When applied to storm sewer discharges and combined sewer overflows, sedimentation is used to provide a moderate detention period to remove a portion of the organic solids and a substantial portion of the inorganic solids that otherwise would be discharged directly to the receiving water. This can prevent the formation of offensive sludge banks. Sedimentation basins have also been used to provide sufficient detention periods for effective chlorination of such overflows [2].

Dual Purpose Detention-Sedimentation. Detention facilities provide flow and/or flood control by retaining, buffering, and attenuating flows; this also provides some level of pollution control by detaining flow long enough for sedimentation or gravity settling to occur. This dual purpose, detention and sedimentation, should be considered carefully during evaluation and design. Through appropriate selection of the design parameters for a facility, the dual detention and sedimentation functions can be optimized. Retrofitting of existing detention facilities can improve their sedimentation efficiency. Whenever possible, facilities should be designed to optimize pollutant removal as well as flow control.

Location in the Sewerage Network

Source Control. Source control methods are used near the source of the stormwater runoff. Source control refers to facilities and practices, used to mitigate stormwater or CSO adverse impacts, which initiate corrective action close to the origin of the stormwater runoff. Inlet control, onsite detention storage, and retention storage are the main categories of source control options.

Inlet control refers to methods used to regulate the flow at the inlet to the sewer system. The detention volume is created by controlling the outflow from specially prepared areas such as horizontal roofs, parking lots, industrial grounds, or other impervious areas.

Cases where the detention takes place at or near the source of the runoff but prior to its entry into the sewer system are referred to as onsite storage. Local handling and controlled release typify onsite storage. Generally, onsite storage is used for detaining the runoff from one or more pieces of real estate. The runoff usually has been transported only a short distance before it reaches the detention facility. Typical onsite storage facilities may be in the form of a diked area, ditch, pond, lined basin, or underground vault with a basin as the inlet.

Retention storage, utilizing the ability of the soil to store or transport water, is another form of source control. With this type of facility, the stormwater is allowed to infiltrate into the soil and percolate down to the groundwater table. In areas unsuitable for infiltration, the stormwater can be directed into specially built storage excavations in a permeable stratum below the ground surface. From these, the water can then percolate into the soil. Retention storage facilities may include percolation basins, drainage swales, dry wells, trenches, porous pavement, and ponds.

In-System Control. In-system storage includes controlled storage in the pipe or drainage channel network and/or in tank or tunnel storage. Detention of stormwater runoff or combined sewer overflows may be accomplished by two methods of in-system control: (1) inline storage, and (2) offline storage.

Inline Storage. Inline storage is the use of excess capacity ordinarily found in a sewer system and/or artificial storage facilities (basins, channels, tunnels, etc.,) to store stormwater runoff or combined sewer overflows. The term, as used here, denotes that the facilities are emptied by means of gravity. A characteristic function in the case of inline storage is that the detained flow interacts directly with the flow which is being transported further in the system. Inline storage is a particularly attractive option in large cities, especially those with large, flat drainage channels or combined sewers for CSO and urban stormwater runoff.

Inline storage is provided by damming, gating, or otherwise restricting flow passage to create additional storage by backing up the water in upstream conduits, channels, tanks or basins. Although simple regulators can be used to generate inline storage, some inline facilities can require a sophisticated monitoring and control system to maximize the storage effectiveness. Although inline storage has usually been applied to combined sewer systems, it is apparent that it also can be used for separate storm sewer systems. The objective of inline storage, when applied to combined sewer systems, has been to maximize the volume of flow directed to a downstream treatment plant while minimizing the overflow frequency, volume, and associated pollutant load to the receiving water.

Inline storage systems have been developed and implemented with varying degress of success, for example, in Seattle [3], Minneapolis-St. Paul [4], Detroit [5], New York City [6], and Switzerland [7]. In the case of New York City, most of the storage is drained by gravity while the remainder is dewatered by pumping.

Offline Storage. Offline storage refers to those cases where the sewer system, through some form of overflow or pumping arrangement, releases flow in excess of some predetermined rate to special storage facilities. Such storage may be in a mined labyrinth, lined or unlined tunnel, cavern, or basin. In this case, the storage facilities are either filled by pumping to storage and emptied by gravity or filled by gravity and emptied by pumping the stored flow back to the sewer system after the storm. In offline storage, the detained flow is temporarily withheld from all further transportation in the pipe network. Often, there is no interaction between the stored water and water which is transported within the system. The objective of offline storage is similar to that of inline storage except that, in many cases, only the initial, heavily polluted flow is captured.

Offline storage facilities have been developed and successfully implemented in Boston [8, 9], Chippewa Falls [10], Saginaw [11, 12], Sacramento [13], Rochester [14], Chicago [15], Milwaukee [16], and San Francisco [17].

Downstream Control. Conceptually, a downstream storage and/or sedimentation facility differs from other detention facilities only in its location in the sewerage system--immediately upstream of the receiving water and/or treatment plant. However, many of its design features are dictated by its function (interception of tributary flows), performance expectations, and environmental setting. Downstream control facilities may be in operation at all times (no bypass option) or in operation intermittently (used at the operator's option). The general application of downstream control facilities is on combined sewer systems, so the volume of wastewater receiving treatment at the dry-weather plant can be maximized. These facilities are usually dewatered during and after storm events as allowed by the available capacity in the sewers, interceptors, and treatment plants. If the storage and/or sedimentation facilities discharge directly to confined waters, the facilities normally include disinfection capabilities.

Downstream storage and/or sedimentation facilities on separate storm drainage systems can be used for flow control and/or sediment and floatables removal. Such a facility can be used to contain first flush runoff from an urban area to minimize the pollutant discharge from additional development. The stored runoff can be discharged slowly to the sanitary sewer system for treatment before discharge to the receiving water. This approach allows the pollutant mass discharged to be held constant or reduced even though the pollutant mass in the runoff may be increased due to the additional urban development. The flow discharged from such a facility, after the storage has been filled, usually does not require disinfection since domestic sewage is excluded from separate storm drainage systems.

Downstream control facilities include lined basins, open ponds, tunnels, caverns, and buried tanks.

Technical Configuration

The technical design of retention, detention, or sedimentation facilities is strongly affected by local conditions. The required volume can be created in a number of ways. There are three principal places to provide the volume: (1) above ground, (2) at ground level, and (3) underground. The following are brief descriptions of many of the most common methods for providing the required volume.

Percolation Basins. Stormwater runoff is conveyed to a suitable open area, which may or may not be covered by vegetation, where the water is allowed to percolate into the ground. In urban areas, the percolation basins may be specially built excavations below the ground surface where the soil is permeable.

Drainage Swales. Drainage swales adjacent to roadways without curbs or in residential areas can be planned to retain runoff. Such swales must be away from houses and side yards so that swampy areas do not develop.

Dry Wells. Dry wells can provide a means of storage as well as significant discharge or dissipation potential in permeable soils. Dry wells should be deep enough so that possible seepage downhill does not create a problem. Provision should be included to minimize siltation and clogging of the permeable soil strata to avoid significant impairment of infiltration capacity [18].

Trenches. Perforated drain pipe or open graded rock fill with the use of filter cloth in a lateral trench can be used for below ground disposition of stormwater runoff. Silt and debris must be trapped before the water enters the trench to prevent clogging of the soil strata.

Porous Pavement. Porous type asphaltic pavement can be used for streets or parking lots where the subgrade has sufficient infiltration and percolation capacity. In cases where the underlying soil does not meet the infiltration and percolation capacity requirements, perforated pipes can be used to collect the stormwater from beneath the subgrade and convey it to another location suitable for infiltration and percolation. Porous pavement is usually used in climates where the ground is not subject to freezing. However, with special design considerations, porous pavement has been used in cold climate locations such as Rochester, New York.

Lakelet System. A series of small water bodies, arranged in series, which provide the necessary storage capacity, can be used to provide sediment control also. Flow introduced into the initial lakelet then flows serially into the remaining lakelets; in effect, acting as a series of storage reservoirs. Sedimentation takes place in each lakelet. Flow through such a lakelet system is usually by gravity with either a gravity discharge or pumped discharge from the final lakelet. A lakelet system can be used on either separate storm drainage or on CSO (as in Mount Clemens, Michigan [12]).

Blue-Green Storage. Stormwater storage in urban drainageways traversing roadways utilizes the roadway embankments as dams and control structures. The structures generally pass small flows unimpeded while ponding occurs when the flow exceeds the pass-through rate [18].

Rooftop Ponding. Horizontal roof surfaces can be used to detain stormwater flow. Such roofs are common for industrial, commercial, and apartment buildings. Building codes in many parts of the country specify that roofs be designed to support snow loads or other live loads. The detention is controlled by a simple drain ring set around the roof drains. As stormwater begins to pond on the roof, flow is controlled by orifices or slits in the ring; extreme flows overflow the ring to prevent structural damage to the roof.

Parking Lots and Streets (Major-Minor Flooding). Stormwater can be detained on paved parking lots by shallow basins or swales. The parking lot should be graded to create multiple storage areas like saucers. At each low point, a catch basin or inlet is used to control the outflow. The outflow control can be accomplished either by restricting the size of the outlet pipe or by using a special cover with drilled holes. The arrangement of the ponding areas within the parking lot should be planned so that pedestrians are inconvenienced as little as possible.

In what can be termed major-minor flooding, the curbline along streets and roads can be used to store stormwater. This storage can be developed by restricting the rate of entry of flow into catchbasins and inlets at the curbline. The stormwater then ponds along the curbline and extends out into the street. The desired depth of the water at the curbline and the spread of the water on the pavement are often criteria for spacing the inlets. It is often possible to allow a greater depth of storage along rural roads than along urban streets. This method of allowing minor flooding within the drainage area can be used rather than inline storage or added treatment capacity to prevent major flooding downstream in the sewer system or receiving water.

Pedestrian Plazas and Malls. In heavily urbanized areas, effective stormwater detention can be incorporated into the design of pedestrian plazas, malls, and other similar type developments. The ponding requirement can be accomplished using shallow depths. Outlet control devices must be checked and maintained frequently to avoid flooding problems and to avoid public inconvenience.

Dry Ponds. Permanent ponds are frequently used for surface storage of stormwater runoff and combined sewer overflows. Dry ponds are small to large depressions, constructed by usual excavation and embankment procedures, that provide for controlled release of impounded water but do not retain water between storms. They can be made to fit well into small developments because they can be designed and installed as small structures.

Athletic fields can be incorporated into permanent dry ponds used for separate storm drainage storage. Soccer or football fields, baseball diamonds, and tennis courts can be made part of a dry pond. The pond can be used as a recreation area for the surrounding community when it is not in active use for stormwater storage.

Wet Ponds. Wet ponds are similar to dry ponds but with additional temporary storage above the normal pool elevation and with provision for controlled release. They are effective in reducing stormwater runoff and channel erosion and have the added advantages of providing water recreation opportunities and of increasing local land value [19].

Check Dams. Check dams or other streambed structures intended to impede and to pool runoff in open channels may be used for areas that contribute to high levels of stormwater suspended solids concentrations. Not only are the adverse impacts of runoff mitigated by the reduced flowrates and lengthened time of flow concentration, but erosion and sediment control can be improved [18].

In-Pipe Storage in Existing Sewers. Because storm sewers and combined sewers are designed to convey maximum flows occurring, say, once in 5 years (50 to 100 times the average dry-weather flow), during most storms there is considerable unused volume within the conduits. This unused volume can be used to detain flow so the peak discharge rate is reduced or so that additional flow is directed to treatment facilities. Various types of regulators can be used to develop this in-pipe storage volume.

Pipe Bundles. The term "pipe bundle" refers to detention facilities that are designed in the form of a number of parallel pipelines with large diameters.

The pipe bundle detention facility is usually connected in series to the sewer network, which means that all of the flow passes through the facility. As a special case, the detention can take place in a single oversize conduit.

Concrete Basins. Concrete basins probably are the most flexible and the most commonly used types of installations. The technical configuration of such basins can be tailored to fit conditions at a given site. Concrete basins can be connected both in series and in parallel with the sewer network. The basins can be above ground, at ground level, or below ground as needed to fit the site. Outflow control devices can include pipes or orifices, weirs, proprietary devices, or pumps as required.

Tunnels and Caverns. Tunnels and caverns can be used as transport, detention, or sedimentation facilities for stormwater and combined sewage. Tunnels may be either inline or offline facilities while caverns (mined labyrinths) are usually offline facilities. Tunnels may be lined or unlined depending on their location with respect to the groundwater table and the geological conditions. Depending on the depth of the tunnel, construction may be by cut-and-cover methods for shallow tunnels or by underground heading for deep tunnels.

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Section 4

SYSTEM PLANNING, DESIGN PROCEDURES, AND INTEGRATION

The multivariable and complex nature of stormwater management assessments make systematic approaches essential. Toward this end it is necessary that system planning be undertaken to ensure that the solution to one problem will not create other problems in the future. The system planning should include an upgrading plan or master plan for the entire system. Usually, the most economical and effective stormwater management system consists of a combination of facilities and techniques integrated into an overall pollution control plan. Storage and/or sedimentation facilities are or should be the backbone of such an integrated plan. They provide inexpensive, effective, and flexible stormwater control that can be constructed singly, in series, or in combination with more advanced techniques as needed. The use of storage and/or sedimentation facilities can be optimized with respect to dual use, as a tradeoff with treatment facilities, multiuse (aesthetic, erosion control, recreation, irrigation, etc.), and augmentation of existing combined sewer overflow treatment facilities.

Of course, stormwater is usually only one of several pollution sources in an urban area. The compatability of various stormwater control techniques with other pollution control facilities and of pollution control with flood control facilities must be considered when developing a unified pollution control plan. The advantages of an integrated control plan include lower overall price and system flexibility. Integration of several small control facilities into a pollution control plan for a developed area is often easier and less costly to construct than a single, large facility. Sites for smaller control facilities are more easily found. Also, additional facilities needed for stormwater control from adjacent developing areas can be integrated into the plan as development occurs.

A multiunit control system allows the level and type of control to be matched to the catchment. For instance, storage and/or sedimentation basins may be used to capture first flushes in areas with combined sewers, or chemically assisted sedimentation may be needed for control of toxic runoff from an industrial area. A multiunit control system retains its flexibility. Additional units or control techniques may be added if the runoff characteristics or available assimilative capacities change. Existing flood control facilities also may be retrofitted to enhance pollution removal in integrated systems. An integrated systems approach allows the addition of new facilities in adjacent areas as the areas are developed, as well as the incorporation of new or improved types of pollution control facilities.

An integrated system approach also allows staging of the implementation or construction of the pollution control facilities. For example, the least expensive (but cost-effective) option, storage and/or sedimentation, can be implemented before going on to a more costly option in a phased program. The use of a source control option is most often less expensive than downstream control, while also helping to alleviate problems in the pipe or drainage channel system downstream. Dual use of an existing pipe network or drainage channel system with bleeding of the stored volume into an existing treatment facility could be implemented prior to or in place of offline storage or downstream control. The development of downstream storage and/or sedimentation options (either with or without utilizing existing treatment facilities) could be the next option. The use of such an integrated system approach can be applied to both separate storm drainage and combined sewer systems.

In this section, it is assumed that the master planning has been completed and that a decision has been reached to include a storage and/or sedimentation facility as part of that plan. Planning concepts, methodologies, and tools common to all storage and/or sedimentation applications are introduced in this section. The need for goal setting and realistic appraisals of options as forerunners to design are stressed.

The role of storage and/or sedimentation in an integrated stormwater management plan is also discussed in this section. The concurrent growth of stormwater control systems and urban areas is examined. Retrofit of existing flood control and drainage facilities to maximize pollution control is discussed. Examples of urban stormwater control are described to illustrate the several points.

SYSTEM PLANNING

Once the master plan or upgrading plan for the system has been completed, detailed planning for a storage and/or sedimentation facility can begin.

Conditions for Planning

Before beginning the detailed planning of a storage and/or sedimentation facility, the general planning conditions that prevail for the facility must be identified. Among the questions to be answered are the following:

- Is storage and/or sedimentation the best solution for dealing with the problems involved?
- Within what geographic region will the storage and/or sedimentation facility be located?
- What type of wastewater will enter the facility?
- What is the goal of the planned facility [1]?

Until the planning conditions are identified and storage and/or sedimentation is concluded to be the most suitable solution, detailed planning for any storage and/or sedimentation facilities should not begin.

Establishment of Goals

Storage and/or sedimentation can be accomplished for different lengths of time, all the way from minimal noticeable detention effect to complete retention of the flow. For planning of such facilities to be meaningful, it is a necessity that the goal(s) for the facility be established.

The goal or goals established are usually based on the overall effect desired for the facility. For example, this can be done in one or more of the following ways:

- The facility should not be overloaded more than a certain number of times per year.
- The volume of wastewater spilled during one year should not exceed a given volume.
- The volume of wastewater spilled on each individual occasion should not exceed a given volume.
- The annual mass emission for a selected pollutant should not exceed a stated value.
- The number of violations of a water quality standard or attainment of receiving water beneficial uses should not exceed a stated value.
- A specified minimum detention time for a particular stormwater runoff rate from a given storm shall not be exceeded.

When the goal or goals have been selected, the storage and/or sedimentation facility can be given the dimensions required to meet the goal. However, the goal established often cannot be used directly in volume determination but must first be transformed into dimensional-design criteria. Examples of such dimensional-design criteria include:

- The facility should be able to contain the flow caused by rain with a certain given statistical recurrence time.
- The facility should be able to contain the flow caused by a certain selected combination of real rains.
- The facility should be able to handle a certain specified flow situation [1].

To design storage and/or sedimentation facilities on a performance oriented basis generally requires extensive calculations. To check whether the performance criterion is satisfied, it is usually necessary to perform continuous simulations of the facility for long periods of actually measured rain. Models available for this continuous planning are discussed later in this section.

Planning Methodology

When storage and/or sedimentation is included as a suitable solution to the established goals, detailed planning of the storage and/or sedimentation facility begins. The use of a defined planning methodology to guide the process ensures that all tasks in the process will be included. Typical design methodologies for source control options, in-system control options, and downstream control options are shown in Figures 9, 10, and 11, respectively.

The basic planning methodology includes the following steps:

- Identify functional requirements
- Identify site constraints
- Establish basis of design
- Select storage and/or treatment option
- Estimate costs and cost sensitivities
- Check that facilities satisfy objectives
- Refine and complete or modify and repeat

The major items to be identified in this procedure are:

- The technical configuration of the storage and/or sedimentation facility(ies)
- The exact location(s) of the installation(s)
- The storage volume(s) required
- The cost of the facility(ies)

Many of these factors are dependent upon one another. To these must be added external factors such as design considerations (flexibility, reliability, management consideration, land requirements, etc.) and environmental assessments (environmental impact, public health effects, social impact, economic impact, etc.). All reasonable alternative solutions should be developed and analyzed as part of the planning process. As stated by Poertner:

> Specific plans are usually oriented only toward various portions of the drainage network; they should always address important relationships to regional land development and resources management. As a study progresses, the periodic need to investigate data and refine the planning work will usually become evident. Thus, the planning process is iterative [2].

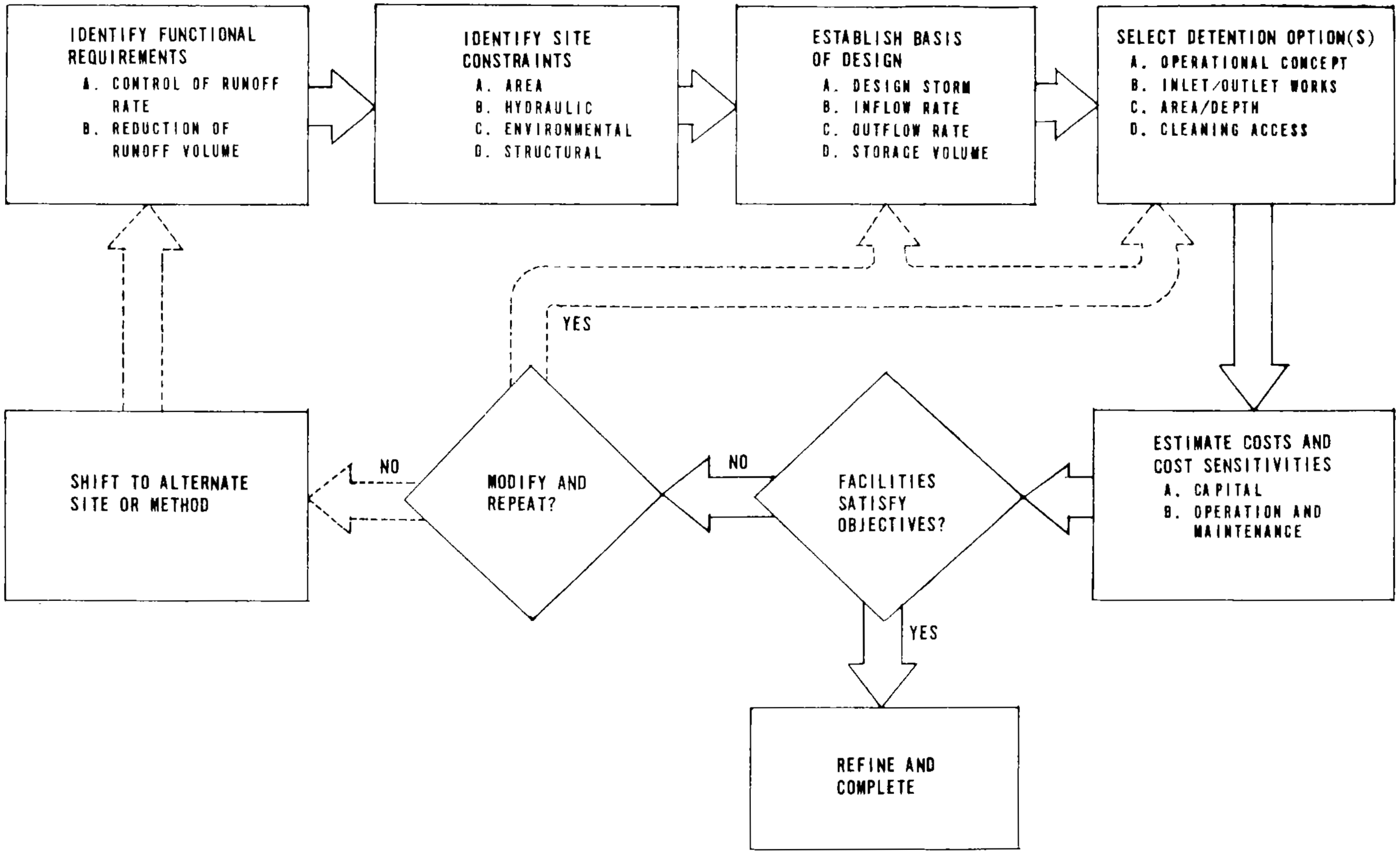

Figure 9. Source control design methodology.

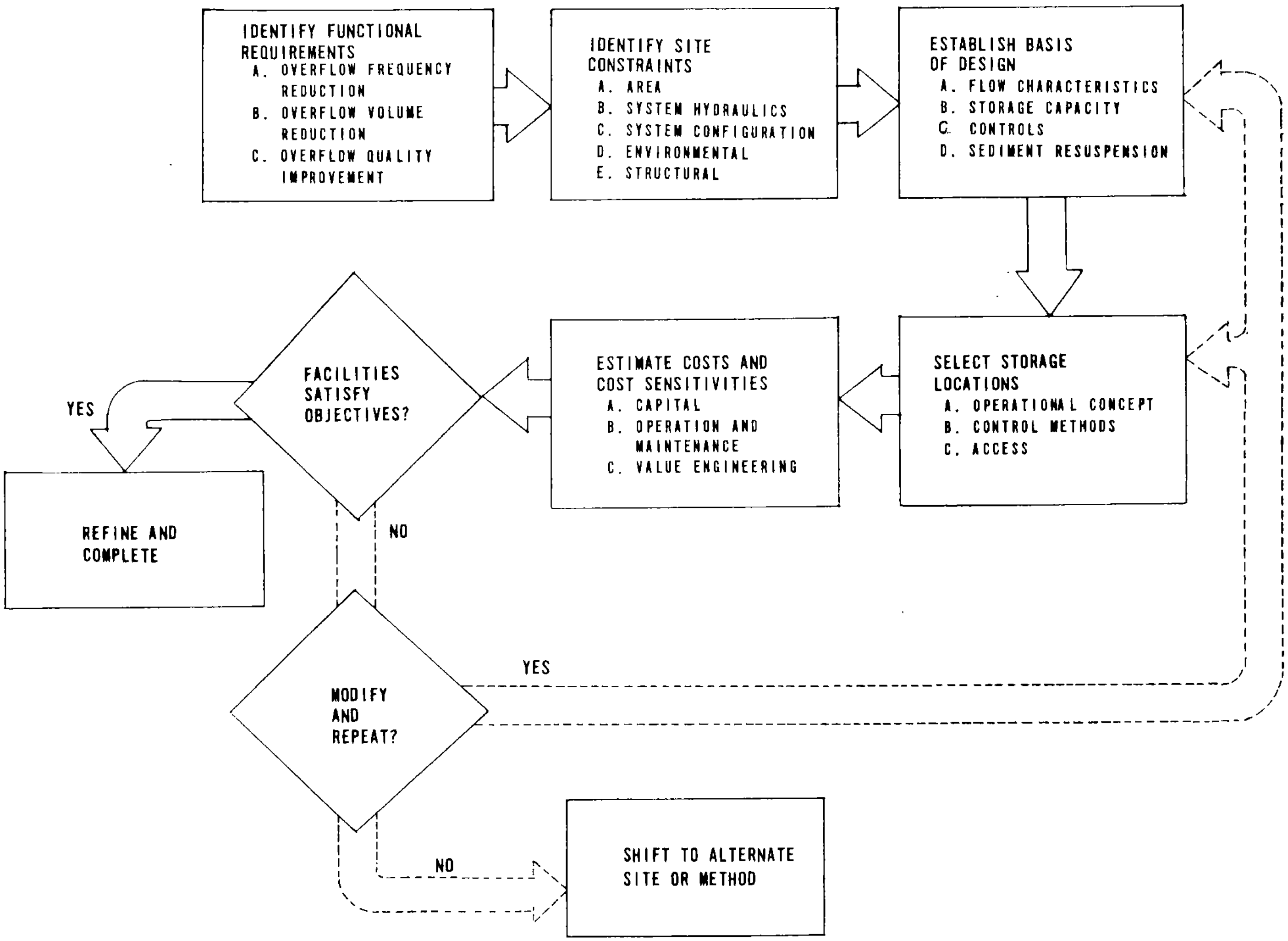

Figure 10. In-system control design methodology.

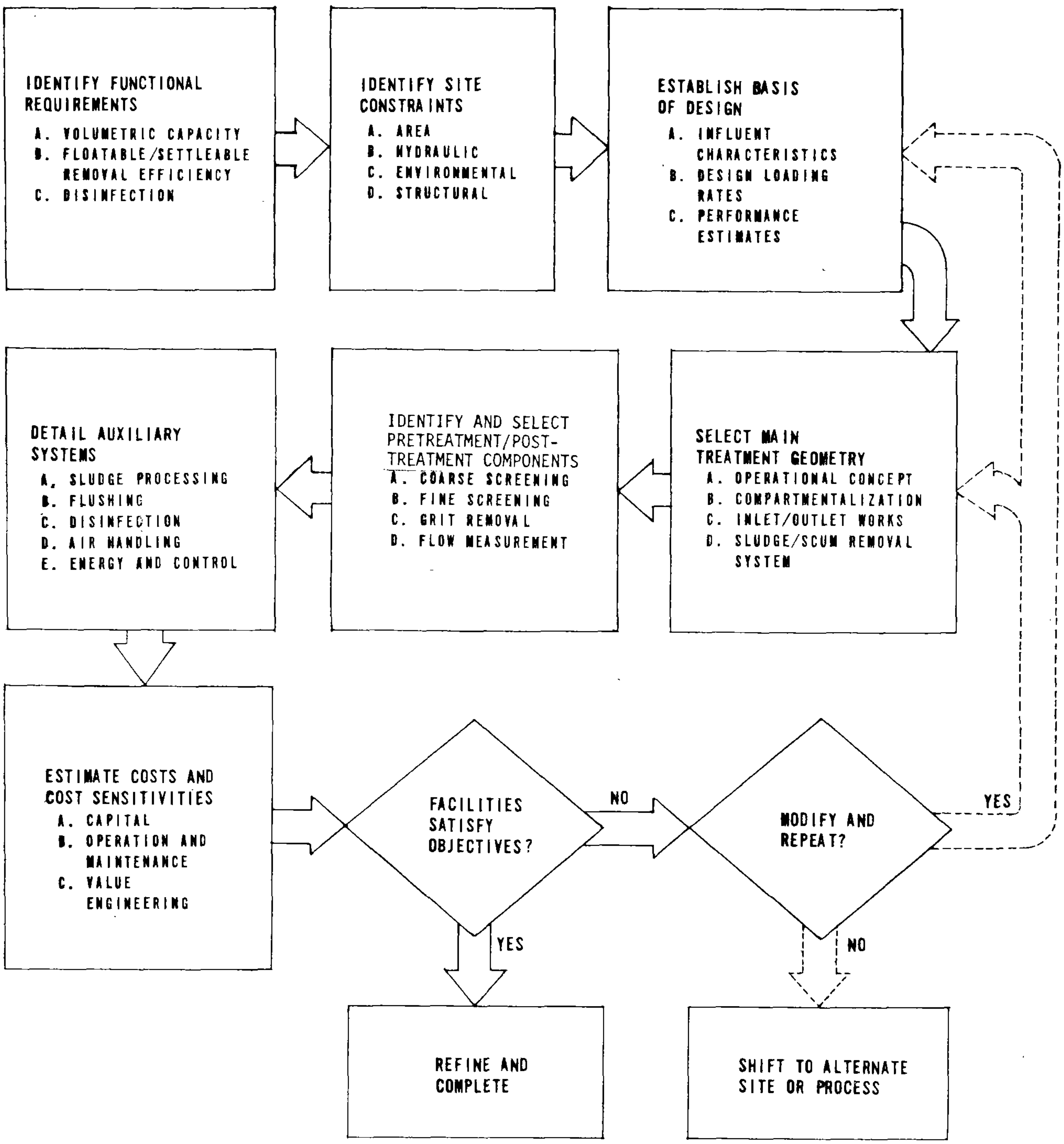

Figure 11. Downstream control design methodology.

Because the planning process is iterative (as shown in Figures 9, 10, and 11), numerous alternative storage and/or sedimentation locations and/or facility sizes may need evaluation. The next step is to select the required storage and/or sedimentation volume.

Cost Optimization Methodology

The methodology used to evaluate the optimum cost of storage and/or sedimentation facilities depends on the purpose of the facility: flow control only, or a combination of flow control and pollutant reduction. In the case of a facility for flow control only, a mass-diagram method similar to that for water supply reservoir sizing should be used. Facilities for flow control and pollutant reduction should use a production theory approach as described by Heaney, et al. [3].

Mass-Diagram Method. A graph of the cumulative runoff and treatment plotted against time is known as a mass-diagram or flow-mass curve. It is the integral curve of the hydrograph which expresses the area under the hydrograph from one time to another. A mass diagram permits a simple graphical inspection of the entire runoff record or any portion of it for determination of either (1) the reservoir capacity required to produce a specified treatment rate (outflow), or (2) the treatment rate which can be expected for a given reservoir capacity. More detailed information is presented by Linsley and Franzini [4] or Chow [5].

Given the series of storage values for the period of record, a statistical analysis of the arrayed storage values can be prepared on a probability plot. A plot of storage capacity with respect to the cost associated with development of that storage capacity should be prepared also. The design storage capacity can then be selected based on some reasonable return frequency and the cost associated with that capacity.

In addition, to meet discharge quality limitations that may be set by regulatory agencies, the storage and/or sedimentation facility may have to be augmented by added treatment facilities. The facilities required can range from the limits of maximizing the storage volume so that no new treatment capacity is required, all the way to providing sufficient additional treatment capacity so that no storage is required. However, the optimum from a cost standpoint usually falls somewhere between these two limits where storage is combined with additional treatment. The optimum combination occurs where the sum of the cost of the required storage and treatment facilities needed to meet the discharge limitations is a minimum.

Production Theory Method. Using the economic principles of production theory, a series of computations can provide an optimized total annual cost for combinations of storage and treatment providing various levels of runoff and/or pollutant control [3]. A graphical representation of this methodology is given in Figure 12. For different combinations of treatment rate and storage capacity (expressed as the depth of runoff contained over the entire drainage area), the isoquant curves in Figure 12 represent equal degrees of

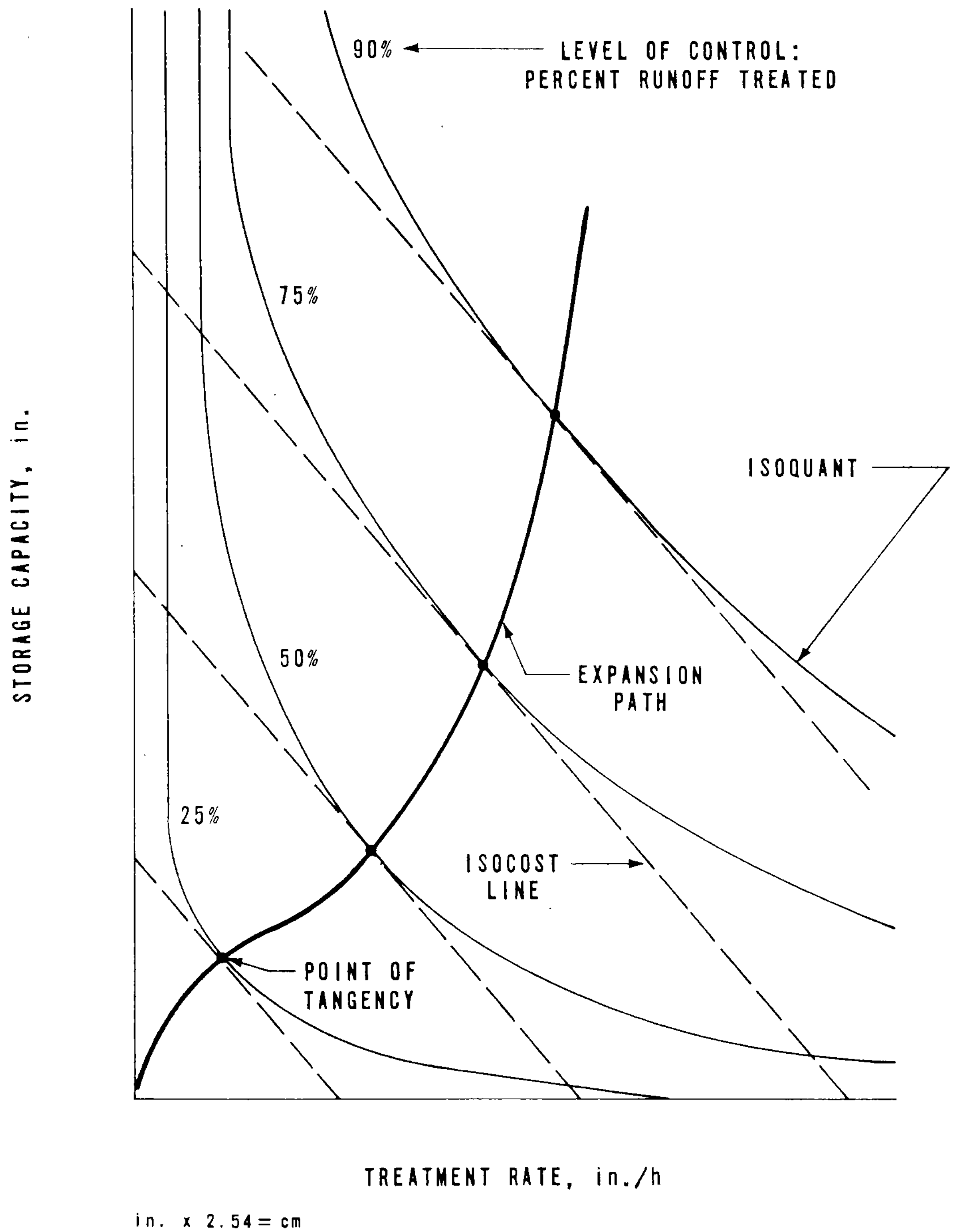

Figure 12. Determination of optimal combination of storage and/or treatment alternatives.

treatment (can be expressed in terms of percent of the runoff treated, percent of pollutants removed, or number of overflows per year). Isocost lines represent storage and/or treatment combinations which may be implemented at the same total cost. The point of tangency between an isoquant curve and an isocost line represents the most economical combination for a given degree of treatment. The optimum combination for any degree of treatment can be found from the "expansion path" through all tangent points.

Storage Volume Determination Methods

A common problem found in the analysis of stormwater management options is to determine how various possible combinations of storage and/or sedimentation capacity will affect runoff control. For example, in a combined sewer system it may be desirable to determine how the effectiveness of an existing treatment plant may be improved by providing storage ahead of the plant. In a currently undersized storm sewer system, it might be desirable to know how the frequency of flooding might be changed by providing various small amounts of storage on the watershed or in the pipes themselves by regulating devices. In a new system, it may be desirable to know the combination of storage and pipe or treatment capacity that will provide the desired level of runoff and pollution control at minimum cost.

The dimensions of such storage and/or sedimentation facilities are determined by the type of facility involved. Depending upon the type of facility and size of the area, a number of approach methodologies are available for determining the required volume. Listed in order of the easiest and simplest for a small area to the most complex for a large area with a complex sewer system, the methodologies are:

- Desktop hand computations
- Statistical analysis of rainfall and flow data
- Simple, continuous simulation of stormwater systems
- Detailed, continuous or single event simulation of stormwater systems

The application of each of these approaches to storage and/or sedimentation volume requirement determination depends upon the size and complexity of the drainage area and/or sewerage system. For small drainage areas with uncomplicated sewer systems, the use of desktop, hand computational methods such as the unit hydrograph approach, reservoir routing, or SWMM Level I [6] for determining the effect of storage on the runoff is usually sufficient. The desktop analysis approach can be used for larger areas when limited rainfall or runoff data availability prevents more detailed analysis. Whenever possible, local rainfall, streamflow, or sewer flow records should be used to verify the desktop analysis or as input for more detailed analysis methods. Information on the unit hydrograph and reservoir routing approaches is available in most hydrology texts [7, 8].

In recent years, mathematical techniques have been developed to reduce the amount of computer work necessary for analysis of stormwater management systems. Methods using statistical characterization of runoff, overflow, and pollution events for preliminary sizing of storage and/or sedimentation facilities for small or simple systems have been developed by DiTorio, Hydroscience, Driscoll et al., and Howard et al. [9, 10, 11, 12]. These methods develop the required statistical information from historical rainfall data and present the levels of control estimated for various storage and/or treatment combinations.

More complex systems can be modeled using EPAMAC, STORM, or SWMM-Version III [13, 14, 15]. These are continuous simulation models. STORM and SWMM-Version III can also be used for single-event storms simply by using a shorter time step. The new Storage/Treatment Block of SWMM-Version III can be run with most runoff simulators (e.g., EPAMAC, STORM, and SWMM-Version III). This model allows the user to input the required relationship between storage volume and outflow rate similar to that for a reservoir routing problem. This approach, permitting the user to best approximate the desired functional relationship, simplifies the model and allows a simulation of a wider range of reservoir geometries and operating policies. Pollutants are characterized by their magnitude (i.e., mass flow and concentration and, if desired, by particle size and specific gravity distributions). Describing pollutants by their particle size and specific gravity distribution is especially appropriate where small or large particles dominate or where several storage and/or treatment units are operated in series. Also, if several units are operated in series, the first units will remove a certain range of particle sizes, thus affecting the performance of downstream units. This model, coupled with site specific settleability and solids characterization information, can be used to make a thorough evaluation and design of retention or detention facilities.

Dynamic wave models, such as EXTRAN, may be needed to design inline storage systems where surcharging and other hydraulic effects are important [16]. This is probably the most sophisticated channel/pipe flow routing model available in the public domain.

Information on numerous runoff models can be found in recent EPA reports [17, 18]. These include ILLUDAS (Illinois Urban Drainage Area Simulator), USGS's DR3M, and SOGREAH's CAREDAS.

Thus, the size of the tributary area and the complexity of the sewer network combine to determine the detail required for sizing and designing the storage and/or sedimentation facility required.

Effect of Storage and/or Sedimentation

To evaluate the different storage and/or sedimentation alternatives, the effect achieved by each alternative must be compared. The term effect can in principle be thought of as the degree of fulfillment of the goal or goals established for the facility. The evaluation of which technical solution is most advantageous is usually arrived at through a cost-effectiveness comparison. Thus, the best apparent alternative should be the facility that both meets the technical goal(s) established and has the least cost.

THE INTEGRATION PROCESS

The process of integrating stormwater control into a pollution control system involves initial planning where existing facilities are identified and goals are determined. Additional steps then involve selecting control methods that are both applicable and compatible to the existing facilities and established goals. Each of the steps is briefly described.

Identify Existing System and Needs

The first step in the integration process is the identification of existing system components and function. This information is the basis of plan development and determination of control methods that are most appropriate for the system.

The watershed characteristics and existing facility components are examined. Information on the land uses of the watershed(s) and acreage of each is used to evaluate the potential for development and to estimate the runoff quality and quantity.

Identification of existing facility location, function, and capacity is required. The compatibility of existing facilities with proposed control methods must be determined. The potential for retrofits with the existing facilities, and whether the existing facilities are in suitable condition must be established. Separate storm and sanitary or combined sewer line location, treatment facility types, and capacities will strongly influence the location and points of interconnection for new facilities.

Establish System Needs

After identifying the existing system and conditions, the next step is to establish additional flood and pollution control needs. Plans should encompass the entire urban wastewater control system including both dry- and wet-weather facilities, and both developed and undeveloped areas. Planning over a whole system enables the use of all resources available. By examining the potential from undeveloped areas, the system can be planned to expand and accommodate future needs much more easily.

Identify Applicable Control Alternatives

The engineer or planner, at this point in the integration process, should be familiar with the existing system and the goals of stormwater management. Alternative control methods should be examined, on the basis of the three factors: (1) physical limitations, (2) effectiveness, and (3) institutional limitations.

Physical limitations may exclude some storage and/or sedimentation methods. Steep slopes or extensive development may preclude the use of retention ponds. In general, older developed areas impose more constraints on the number of options available because the sewer network(s), and other underground utilities and structures are already in place. The same may also be true in developing areas, but usually to a lesser extent. Land availability will also limit the control method options. Limited land availability tends to preclude open ponds, but storage and/or sedimentation basins would be applicable because the structure could be built to physically support a second use above it. A list of the more common control methods and the physical situations favoring their selection are shown in Table 8.

Table 8. STORAGE AND/OR SEDIMENTATION CONTROL METHOD VERSUS FLOW OR QUALITY APPROACH

	Flow attenuation	Quality improvement
In-system storage	X	X
Rooftop ponds	X	-
Plaza ponds	X	-
Parking lot ponds	X	-
Storage/sedimentation basins	X	X
Dry detention/retention ponds	X	X
Wet detention/retention ponds	X	X
Major-minor flooding	X	-

Note: Flow attenuation can be considered to provide some quality improvement through flow reduction, flow redistribution, and pollutant load redistribution.

The control methods also vary as to their effectiveness in reducing pollutant concentrations and mitigating storm volumes. Retention ponds for separate storm drainage are highly effective for both flow attenuation and pollutant removal, since the flow is totally contained. Thus, neither the flow nor the pollutant load is transmitted downstream as long as the available volume of the pond is not exceeded. Flow in excess of the pond capacity is discharged through the overflow structure (see Figure 8). Thus, a retention pond provides both flow control and pollutant reduction for all flows in excess of pond capacity by acting as a storage and sedimentation facility during the overflow period.

Source control may be used to attenuate storm flow for both separate storm drainage and combined sewer systems. It is somewhat less effective than retention ponds for pollution control; the pollutants remain available for later resuspension and transport unless they are physically removed from the source control facility. In some cases, source control provides only flow attenuation since the total volume (including the pollutants) is eventually discharged to the sewer system. This redistributes the pollutant load (reducing shock loads) and increases the volume that is directed to treatment.

In-system and downstream controls offer both flow and pollutant mass discharge rate attenuation for the storm flows during a storm event. In separate storm drainage systems, both the total flow volume and pollutant mass discharged remain undiminished since the stored volume is usually discharged to the receiving water just as before storage was implemented. Shock loads on the receiving water are reduced through redistribution of the pollutant load. If the stored stormwater is bled back to the sanitary sewer for treatment, the flow volume and pollutant load reaching the receiving water are reduced.

In combined sewer systems, in-system and downstream controls provide attenuation and reduction for both flow and pollutant loads. During a storm event, the storage and/or sedimentation facilities provide flow and pollutant mass discharge rate attentuation. Following a storm, the flow and pollutants remaining in storage are released to the sewer or interceptor for transport to the treatment plant for processing. The pollutants remaining in storage at the end of a storm event are usually more than just those associated with the flow volume retained due to the sedimentation that occurs during storage. Thus, the pollutant load reduction resulting from storage during the storm event may be much greater than the flow reduction for that same event.

The effectiveness of the various storage and/or sedimentation control methods is shown in Table 9.

Table 9. STORAGE AND/OR SEDIMENTATION CONTROL METHOD VERSUS PHYSICAL AND EFFECTIVENESS LIMITATIONS

		Effectiveness	
	Physical/environmental	Flow	Quality
In-system storage	Extra capacity must be present inline or offline	Proportional to capacity available	Some, for combined sewer systems*
Rooftop ponds	Flat roof structures	Yes for peaking flows	Some, for combined sewer systems
Plaza ponds	Land area for development	Yes for peaking flows	Some, for combined sewer systems
Parking lot ponds	Public inconvenience	Yes for peaking flows	Some, if street sweeping program in effect
Storage/ sedimentation basins	Land use conflicts	Potentially high depending on mode of operation	Yes--up to 60% SS removal; other parameters vary
Dry retention ponds	Large space requirement, flat terrain	Yes, 100%	Yes, 100%
Wet retention ponds	Large space requirement	Yes, 100%	Yes, 100%
Dry detention ponds	Large space requirement, flat terrain	Yes for peaking flows	Yes--up to 60% SS removal; other parameters vary
Wet detention ponds	Large space requirement, flat terrain	Yes for peaking flows	Yes--up to 60% SS removal; other parameters vary

*Some quality improvement for separate storm sewer systems where the stored storm runoff is bled back into the sanitary sewer system for conveyance to a treatment facility.

The third factor determining applicability involves institutional limitations. The means of implementation, operation, maintenance, and financial support are crucial to the applicability of a management system.

The institutional authority must exist to ensure the implementation of the system and continued system operation.

Funding for implementation, operation, and maintenance must be secured. An institutional organization must exist to collect funds, impose fees, or make other necessary financial arrangements. If the cost is to be borne by developers, then the legal and enforcement authority must exist to confirm that the control methods are applied.

Determine Control Method Compatibility

Once the applicable control methods are identified, the final step in the integration process is to assess the process compatibility. Process questions include (1) which control methods have compatible treatment methods, (2) will capacity of existing treatment be exceeded due to installation of control method, and (3) is the method flexible for use in other treatment process trains.

Control method compatibility with treatment processes is an important consideration of the integration process. A chemical treatment process that could upset a biological process should not be used ahead of the latter. Also, pretreatment requirements for the functioning of some processes must be incorporated into a system and could be part of stormwater treatment facilities.

Flexibility of dry- and wet-weather facilities in either combined sewer or separate storm drainage systems can take advantage of the wet-weather facilities as a pretreatment process for dry-weather flows, or as a standby in case of dry-weather facility failure. Wet-weather facilities might also be used as an effluent polishing step during dry weather.

Another important compatibility consideration is the effect of sludge generation by the stormwater management control methods. From a pollution control standpoint, the sludge resulting from the use of storage and/or sedimentation facilities should be removed, whenever possible, from basins, ponds, or pipe networks where it first settles to prevent its resuspension and transport downstream. This would reduce the solids mass load downstream on either additional storage and/or sedimentation facilities or the receiving water. From a practical standpoint, this may create a massive logistical problem of providing access, sludge removal equipment, and transportation for the sludge removed. In most cases, it is most economical to resuspend the settled matter and discharge it to the sanitary or combined sewer for transport to the dry-weather treatment plant where sludge collection, processing, and disposal equipment already exist providing that there is sufficient capacity to handle the additional suspended solids. Onsite removal of sludge is usually practiced only at large, lined open basins where easy access for sludge removal equipment is provided. An integrated facility must include transportation and processing of the solids (either onsite or offsite) in the overall plan.

DESIGN PROCEDURE FOR COMBINED SEWER SYSTEMS

The main steps to be followed in the design procedure for combined sewer systems are (1) problem identification, (2) data needs, (3) determination of the pollution load, (4) identification of the pollutant removal objectives, (5) control optimization, (6) pollutant budget analysis, (7) operating strategy for design, and (8) instrumentation and control strategy for operation. During the design procedure, specific consideration must be given to sludge/residuals removal and disposal from the facilities that provide storage, sedimentation, or both.

Problem Identification

The investigation of stormwater discharges is concerned with two different types of polluted flows--separate stormwater runoff from storm sewers or drainage channels, and combined sewer overflows from sewers containing both storm runoff and sanitary sewage. However, the problems associated with these discharges can be identified as (1) quantity of flow, and (2) quality of flow. Before the design procedure can proceed, it is necessary to identify the problem as either flow related, quality related, or both.

Flow. Flow problems are usually identified with flooding and flood damage. This can be streams overtopping their banks, runoff exceeding the capacity of surface drainage channels or combined sewer inlets, flooding of basements, or flooding of streets and surface areas due to surcharging of combined sewers.

Quality. Overflows from combined sewers can produce serious pollution of local waterways and receiving water bodies. The surcharged sewers often spill their contents into streets, highway underpasses, and basements of buildings. This results in flooding, pollution, health threats, inconvenience, and economic losses associated therewith. The magnitude of the potential effects resulting from CSOs was presented earlier in Table 3.

Data Needs

Comprehensive master planning is required to achieve the goals and objectives of urban combined sewer overflow control. It is most important to gather sufficient data so the number of iterations required for the planning and design of facilities is minimized. Information relating to historic, existing, and future land use; basic hydrology (including rainfall, runoff, vegetation, soils, and infiltration); discharge capacities of existing facilities; impacts on adjacent properties; evaluation of the existing problems; and details of existing master plans for the area are needed. Alternatives can be developed only after analyzing the collected data.

Rainfall. The design of combined sewer systems is usually based on precipitation events having a statistical frequency of occurrence. In the past, statistical rainfall intensity-duration-frequency relationships were used to size the sewers and storage facilities. This approach resulted in sizing for peak flows but did not account for the effects of short intervals between storms or uneven areal rainfall during storms. The use of intensity-duration-frequency relationships should be avoided except for initial rough-cut estimates.

The sizing of combined sewers, storage facilities, and sedimentation facilities should be based on a continuous historical or synthesized rainfall record that is typical of any long-term rainfall record. The actual historical records selected may be based on the hourly intensity, storm duration, total rainfall for the storm, or any combination of these.

For example, rainfall characterization analyses run on historical hourly rainfall records for San Francisco periods of 70 years (full historical record), 4 years (October 1971 through April 1975), and 4 months (October 1972 through January 1973) resulted in remarkably consistent results [19]. These results are shown in Figures 13, 14, and 15, for storm magnitude, intensity, and duration versus frequency, respectively. In all cases, the 4-month record correlated well with the full 70 years of record. Thus, the use of the 4-month record permitted the preliminary screening of a large number of alternatives (storage volume and treatment plant capacity) while using only a modest input of project funds, time, and labor. These shorter periods were judiciously selected after detailed review of the full 70 years of record. The purpose of these selections was to minimize the computer time and cost associated with evaluating the alternatives for storage and treatment capacity combinations.

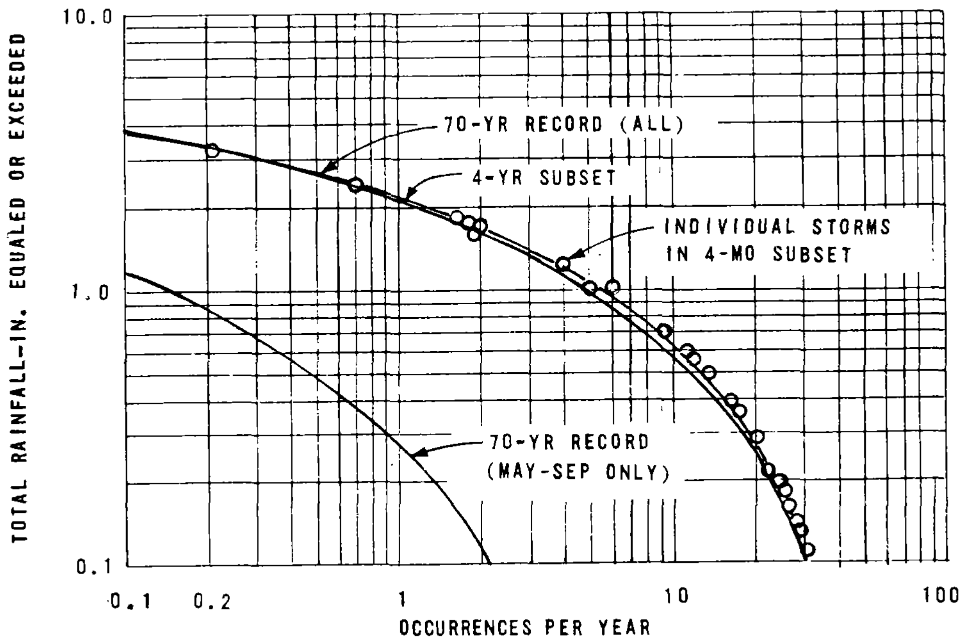

Figure 13. Storm magnitude versus frequency.

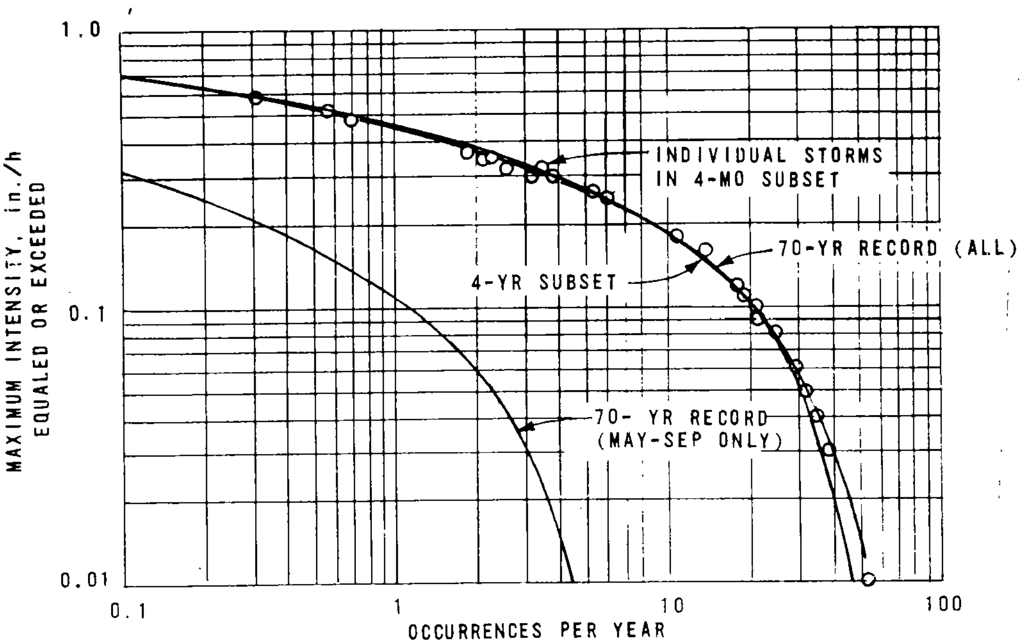

Figure 14. Storm intensity versus frequency.

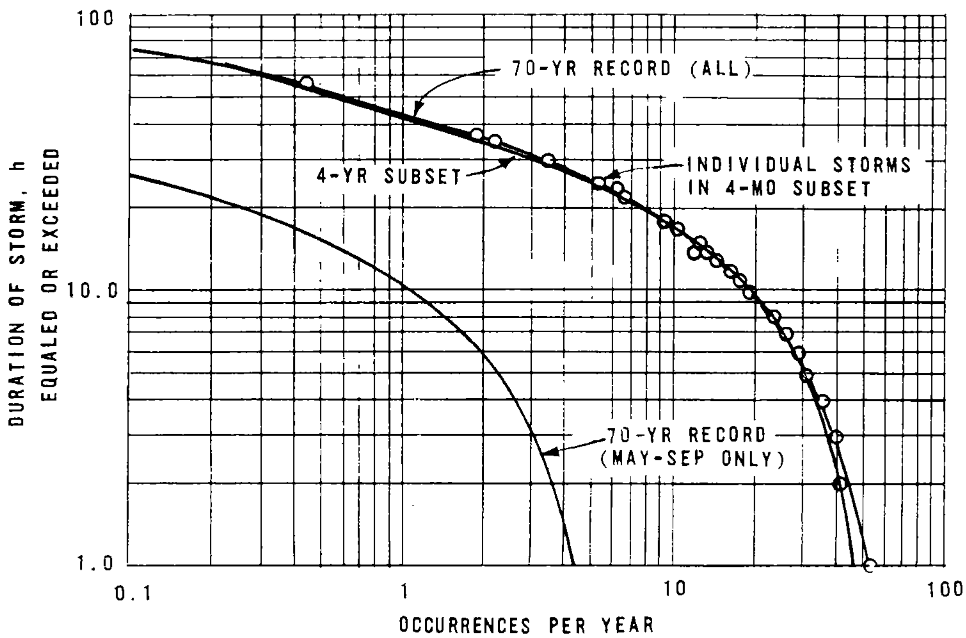

Figure 15. Storm duration versus frequency.

Rainfall data are available from the National Weather Service (NWS). Additional local rainfall data may be available from municipal sources, local flood control or water districts, or utilities.

Flow Records. To accurately assess either the flow or the quality problem (or both), if it is necessary to know what problem or problems were caused by the flow. Use of historical flow records, when matched with the appropriate rainfall data, can show what problems are caused, when they occur, and where they occur (i.e., surcharging of sewers, flooding of basements, overflowing at pumping stations, etc.).

Dry-weather flow records are needed to determine the diurnal flow and quality variations. The stormwater carrying capacity of the combined sewer at any time can be found by subtracting the dry-weather flow from the maximum capacity of the sewer. The pollutant mass load in the dry-weather flow at any time must be added to the pollutant load in any stormwater present to determine the total mass load or the concentration of the total flow at that time.

Wet-weather flow records are needed to determine the runoff coefficients for the tributary area, the response of the sewer system to various rainfalls, and the effect of the storm flow on pollutant loads (i.e., first flush phenomenon). It is important to know the flow response to various rainfall events so that the effects on the sewer system of a design storm or a particular series of historical storms can be predicted. In most cases, the wet-weather flow data available cover only a small portion of the time for which historical rainfall data are available.

Drainage Area Characteristics. The drainage area characteristics influence the volume of flow and the rate of runoff from the tributary area. Pertinent physical characteristics of drainage areas that affect both the volume and rate of stormwater runoff include topography, land use, population density, geology and soils, and size.

The topography can affect the rainfall patterns (i.e., additional rainfall due to orographic lifting) as well as the rate of runoff. The runoff rate is usually increased as the slope of the ground increases.

The distribution and types of land use within the drainage area can greatly affect the runoff. Usually, as the population density increases, so does the percentage of imperviousness in the drainage area. The percentage of imperviousness can be affected by the type of land use also (i.e., parks and recreational, streets and highways, industrial, commercial, and residential).

While the size of the drainage area alone does not determine the runoff volume and rate, it can have a great effect on them. The runoff volume, on a unit area basis, is determined by the other physical characteristics of the area for any given rainfall event. However, the total runoff volume is determined by the unit area runoff and the size of the area. The rate of runoff is also affected by the size of the area. Usually there is some attenuation of the runoff rate as the drainage area increases caused by attenuation of the runoff flow by travel time and by areal distribution of the rainfall. In large areas, the rainfall may not be uniform over the entire area (i.e., thunderstorm cells), thus producing a relative reduction in rainfall intensity for the whole area.

The geology and soils affect the runoff also. The depth, porosity, and type of material determine the rainfall storage capacity before surface runoff occurs. The antecedent conditions (i.e., time since last rainfall, soil saturation, etc.) also affect the runoff rate and volume. The type of soil has a great effect on the amount of suspended solids that is contained in the runoff. Cohesionless soils usually contribute more to the suspended solids load in the runoff than do cohesive soils.

<u>Suspended Solids Characterization</u>. In addition to those solids normally found in sanitary sewage, combined sewer overflows contain solids washed into the sewer system from urban roadways and land areas. High flowrates in the sewers during storm events resuspend solids deposited in the lines, adding additional suspended solids (generally grit and sand) to the solids load. A characterization of the suspended solids, including floatables, in the flow is necessary to determine or estimate the sediment/floatables removal resulting from storage or sedimentation facilities in the system or proposed for the system.

Stormwater from different locations generally has extremely varying properties. Among the factors that are of significance are:

- The pollutant content of the combined sewage
- The proportion of easily settling pollutants
- The particle size distribution in the combined sewage
- The particle volume distribution in the combined sewage
- The density of the combined sewage particles

With regard to sedimentation of combined sewage, it is mainly the content of suspended material in the water which is of interest. Other pollutants (such as BOD, heavy metals, etc.) that may be bound with the suspended material that is settleable can be removed during storage or sedimentation. Therefore, it is important to identify those fractions of the pollutants bound to the settleable solids so that the effectiveness of sedimentation on overall pollutant removal can be estimated. The remainder of the pollutants will be included in the supernatant. The content of suspended material in the combined sewage should be at least 75 to 100 mg/L for sedimentation to be useful; sedimentation is of virtually no value when the suspended solids concentration is below 20 mg/L [1].

The heavier suspended material can represent a quite significant part of the total pollution content of combined sewage. The time required for settling out the coarsest pollutants ranges up to just a few minutes. The limiting value for the separation of the coarser solids in the combined sewage should be set at 5 minutes. This will include particles in the sand range and larger.

The particle size distribution in different stormwaters has a significance on the sedimentation properties of the suspended solids. For combined sewage that has first undergone a coarse separation as described above, the majority of the remaining suspended solids particles are usually found in the 5 to 75 µm range. The number of particles is largest in the 10 to 25 µm interval. The number of particles larger than 40 µm is often smaller than 10 per mL [1].

To estimate how large a quantity of the pollutants will be separated out by sedimentation, the density of the particles must be known. While data in the literature are sparse and contradictory, typical values that have been reported are in the range of 68.6 to 81.1 lb/ft^3 (1100 to 1300 kg/m^3) [20].

The suspended solids should be characterized by both particle size distribution and density. The particle size distribution can be determined from a sieve analysis of the suspended solids. The specific weight of the particles must also be determined. A settling column can be used to determine the settling characteristics of the suspended solids. The procedure and equipment for settling column tests are described by Dalrymple, et al., and Pisano, et al. [21, 22].

Collection System. The configuration of the collection system will determine where storage or sedimentation facilities can be located. The size and slope of the pipes have an effect on the use of the pipes for inline storage. The diameter of the pipe will limit the volume per linear foot of pipe that can be stored; the slope of the pipe will determine the length of pipe available to provide storage without exceeding any surcharge limitations at the control point. The slope also determines the flow velocity which affects the shear forces on solids particles. This affects the suspension or resuspension of the particles.

The overall length of pipes and the pipe density within an area also determine the potential volume that can be stored from any control point within the system. The longer the pipes are and the higher the number of pipes per unit area, the more volume is available for storage at any given pipe slope.

The configuration of the sewer system affects the flowrate and time of travel within the system. For a given unit area, the flowrate increases and the travel time decreases as the number of branches increase providing the sewer slope remains the same.

The type and location of overflow structures or flow regulators or both also affect the inline storage capacity of a collection system. Any storage downstream of an overflow structure will be limited by the depth to which flow can be stored before the level rises to the elevation of the overflow. Inline storage downstream of a regulator must be limited so that the backwater from the storage does not affect the operation of the regulator.

Existing pumping stations can be used to control inline storage upstream of the pumping station. The storage can be effected by controlling the number of pumps operating at any given time. The flow can be stored until the hydraulic grade line upstream reaches a predetermined level (overflow elevation, basement level, maximum surcharge on a sluice gate, etc.) that will not cause aesthetic or economic problems. If a pumping station is used for storage control, the station pumping capacity must be sufficient to pump any flow required to prevent problems upstream.

Determine Pollution Load

Most analyses of pollutant concentration measure the total quantity but do not distinguish between soluble and particulate fractions. Sedimentation computations are based on the particulate or settleable fraction (this should also include floatables since they are removed by skimming as part of the sedimentation process). However, overall removal efficiency is expressed in terms of total quantities of pollutant, which is the most relevant way to express results for control decisions and forms the basis for reporting observed results to be used for comparison with computations.

Therefore, it is necessary to determine the fraction of the total concentration or load which is settleable. This is most conveniently done by determining the fraction of pollutant associated with each of several ranges of particle settling velocities and combining the results to obtain the overall removal. It is also important to know the soluble fraction of pollutant for reasons that are discussed below.

The effectiveness of any storage or sedimentation facility cannot be estimated unless the pollution loads entering the facility are known. This requires knowledge of the various pollutants in the combined sewage as well as the time variation of the mass loading of these pollutants. The pollutants of most common interest are BOD and suspended solids. However, other pollutants of interest usually include one or more of the following: volatile suspended solids; COD; various forms of nitrogen; phosphorus, both total and orthophosphate; heavy metals, particularly lead; and total and fecal coliforms.

Some of these pollutants are most frequently adsorbed onto the suspended solids rather than being in the dissolved form. Thus, it is necessary to determine whether the pollutants of interest in any particular study are associated with the suspended solids, dissolved, or both. If the pollutants of interest are adsorbed onto the suspended solids, it is necessary to determine the fractions associated with the settleable solids and the colloidal solids. This information is needed to determine whether any removal will be affected by use of storage or sedimentation facilities.

The determination of the pollutant load can best be accomplished through a site-specific sampling program. The sampling program should be developed to characterize the quality of both the dry-weather flow and the combined sewage during wet weather for the predesign of an abatement program. The pollutant values are a combination of the sanitary sewage pollutant concentrations and the stormwater runoff pollutant concentrtions. Site specific concentrations that result from this mixture depend on the quality of the two base flows and their proportional mix.

Identify Pollutant Removal Objectives

Once the pollutant loads are known, the pollutant removal objectives can be identified. The removal objectives should be selected in conjunction with the problem identified, quantity of flow or quality of flow or both, and the pollutant loadings.

If the prime objective is storage (i.e., flood control, flowrate control, etc.), the facility should be designed primarily for storage. However, since there is sedimentation associated with any storage facility, the sedimentation cannot be ignored. A means for removing or resuspending the settled solids must be incorporated into the design. Otherwise, the solids would continue to build up over time and reduce the effectiveness of the storage.

If the major objective is the removal of a pollutant that is primarily associated with the settleable solids, the facility should be designed as a sedimentation facility to maximize the removal of the settleable solids. If the prime objective is the removal of a pollutant that is dissolved or associated with the colloidal solids that do not settle, the facility should be designed to maximize the storage volume. In both cases, any settled solids can be removed at the storage or sedimentation facility location or they can be conveyed to a treatment plant for removal. The volume of flow remaining in the storage or sedimentation facility after a storm event should be conveyed to a treatment plant for treatment before discharge.

Control Optimization

The objective of any combined sewer overflow control system should be to maximize the effectiveness of the control system by producing the maximum desired effect for the funds available. The desired effect can be the reduction of a single parameter (i.e., overflow frequency, overflow volume, pollutant mass, etc.) or the reduction of any combination of parameters. In any case, the desired effect must be identified and selected prior to optimization of the control strategy.

The steps involved in the optimization of the control strategy are:

- Select the desired effect(s)
- Identify location(s) and capacity(ies) for storage or sedimentation or both

- Identify location(s) and capacity(ies) for treatment facilities

- Identify the costs associated with each storage, sedimentation, and treatment facility

- Evaluate the effect of changes in the number and size of the various facilities upon the cost of the control strategy

Identify Location(s) for Storage. The maximum size and the location of each existing or proposed storage facility must be determined. For any given site, there is a maximum storage volume that can be physically accommodated. However, any smaller storage or sedimentation facility can be accommodated. The storage or sedimentation or both can be developed through source control, in-system facilities, downstream facilities, or any combination of these facilities.

Identify Treatment Location(s). The maximum size and location of each existing or potential treatment facility must be identified. The size for any treatment facility can vary downward from the identified maximum. Treatment can be provided at existing plants, expanded existing plants, satellite plants, or any combination of these facilities.

Cost Analysis. The cost identified for the storage, sedimentation, and treatment facilities should include both capital and operation and maintenance costs. A range of costs for each facility should be determined based on cost per unit of storage volume, per unit of pollutant removed, per unit of overflow volume reduced, or per unit of treatment capacity as is appropriate for the particular facility.

Optimization. The effect of the variation of size or capacity of the individual facility on the entire system cost should be determined. The combination of storage, sedimentation, and treatment facilities that provides the greatest effectiveness for the acceptable cost can be identified as described previously in this section under Cost Optimization Methodology.

Pollutant Budget Analysis

To determine the overall effectiveness of a storage and/or sedimentation facility, it is necessary not only to determine the fraction of any specific pollutant that is retainied in the facility, or removed as a result of the facility, but also to determine the timing of the removal. In other words, it is necessary to know when and where the removal occurs. A pollutant budget analysis is simply a means of keeping track of the mass of pollutant that is associated with the residuals (sediment) and with the supernatant (overflow or discharge). It is necessary to know not only that the pollutant mass inflow is equal to the sum of the pollutant mass in the residual and the supernatant, but also the time relationship among masses.

Residuals. The sediment remaining in a storage or sedimentation facility depends on the particle size and density distribution as well as the particle volume distribution. It is important to know the time relationship of the residuals accumulation to determine whether it is necessary to continue the sedimentation operation or whether the flow can be passed on downstream untreated. It is possible for the sediment distribution throughout the storm to be such that most of the readily settleable material is included during the early part of the storm. Therefore, the later portion of the flow may be passed on untreated without serious effect on the downstream receiving water.

Specific pollutants of interest may be associated with a specific particle size/density or settling velocity range. The effect of sedimentation on this particle range must be determined so the efficacy of sedimentation for removal of the pollutants can be estimated.

Supernatant. The supernatant (treated discharge or overflow) will contain a portion of the pollutant also. The effectiveness of storage or sedimentation on pollutant removal is determined by the ratio of the mass of the pollutant removed to the total mass of the pollutant included in the flow.

Thus, the pollutant budget analysis is a time-related mass balance for the pollutant or pollutants of interest.

Operating Strategy for Design

At this point, it is necessary to determine an operating strategy for the combined sewer overflow control facilities. The alternatives for the physical layout of the facilities must be selected (both locations and sizes), the pollutant budget analyzed and refined, and an optimization analysis performed. This is necessary to identify the apparent best alternative.

Layout Alternatives. The location of alternative storage or sedimentation or combination facilities must be selected so that a manageable number of alternatives can be evaluated. The facilities should be evaluated based on modular sizes to facilitate the evaluation process. The facilities can be individual facility locations or combinations of locations. The entire combined sewer collection system and treatment facilities should be included in the analysis.

Pollutant Budget Analysis Refinement. The pollutant budget must be reanalyzed for the most attractive alternatives to determine whether they meet the desired goal for pollutant removal. This is necessary to assist in the optimization evaluation.

Optimization. The costs associated with each facility in the combined sewer overflow control system are used to determine the apparent best alternative. The optimization should determine the alternative that provides the greatest desired benefit for the least cost as long as that cost is less than the economic limit established for the project. It may be necessary to run through an iteration process to determine the optimum project.

Instrumentation and Control Strategy for Operation

A means of controlling the storage or sedimentation facility(s) must also be identified. This should include identification of the instruments for control as well as a control strategy. A variety of instruments and control strategies are available. These can include monitoring opacity, use of radar to monitor rainfall, establishment of a rule curve for operation, or remote control from another location.

Opacity. The opacity of the flow in a combined sewer can be used to control flow into or out of a storage or sedimentation facility. For example, as the opacity of the sewer flow increased as a result of increased suspended solids concentrations due to stormwater, the flow can be directed into a storage facility. Later, as the opacity decreases when the stormflow becomes more dilute, the flow into the storage facility can be stopped and the flow directed on downstream.

Opacity can be used in sedimentation facilities to control sediment removal rates or chemical additions. Chemicals can be added to improve the sediment removal.

Radar. Radar is presently used to monitor the location and intensity of rainfall during storms. Radar may be used to help determine the best strategy for sequencing operation of storage or sedimentation facilities on a real-time basis. Radar can be used to provide decision-making information for controlling the drawdown of storage or sedimentation facilities prior to the arrival of a storm or between a series of storms. Radar is not presently used as the sole control for the operation of any specific combined sewer overflow or separate storm sewer storage or sedimentation facility or facilities.

Remote Control. Remote control of combined sewer overflow facilities is common now. It is not necessary to staff each facility as long as enough data for decision making are transmitted from the facility site to the manned control location. The operation of the storage or sedimentation facility can be monitored or controlled from the remote location. A typical example of this is the combined sewer overflow regulator control at Seattle. A series of regulators located throughout the city are controlled from a central location.

Rule Curve. Control facilities can be operated on the basis of a rule curve established from previous experience. A typical example of this is the use of a programmed process controller. A series of control functions are triggered based on a previously established time sequence or liquid level. In this case, the operation of the facility is the same each time it is activated. A rule curve is used most often for remote facilities.

The cost of a combined sewer overflow control or treatment facility (storage, sedimentation, or both) is affected by the cost of the instrumentation and the control strategy selected. This cost must be incorporated into the optimization process.

DESIGN PROCEDURE FOR SEPARATE STORM SEWER SYSTEMS

The main steps to be followed in the design procedure for separate storm sewer systems are (1) problem identification, (2) data needs, (3) determination of the pollution load, (4) identification of the flood control and pollutant removal objectives, (5) control optimization, (6) pollutant budget analysis, and (7) operating strategy for design.

Problem Identification

As with combined sewer systems, the problems associated with separate storm sewer systems fall into two categories: (1) quantity of flow, and (2) quality of the flow. It is also possible that the problem is related to a combination of the two categories.

Quantity. Flow problems are usually identified with flooding and flood damage. Increased urban development upstream of existing storm sewers may contribute increased stormwater flows that exceed the capacity of the existing sewers. This is usually the result of increasing the impermeable area within the watershed. Flooding of streets, public areas, and buildings may result from the increased flow. Receiving streams may flood also as a result of the increased stormwater runoff.

Quality. Urban development can introduce many additional pollutants to the stormwater. Examples of the pollutants may include waste engine oil dumped into catchbasins, paints and solvents, additional suspended solids, and floatables (bottles, cans, styrofoam containers, etc.). These pollutants may create problems within the storm sewer system by settling and creating restrictions or they can create problems in the receiving water.

Data Needs

Comprehensive master planning is required to achieve the goals and objectives of urban stormwater control. The data needed for the comprehensive planning include land use, basic hydrology (including rainfall, runoff, vegetation, soils, and infiltration rates), storm sewer configuration and capacities, impacts on adjacent properties, evaluation of existing problems, and details of any existing master plans. Alternatives can be developed only after analysis of the collected data.

Rainfall. Storm sewers are typically designed based on precipitation events having a statistical frequency of occurrence. This is the same as for combined sewers. The information needed is the same as that described previously for combined sewers in this section.

Flow Records. As with combined sewers, both dry-weather and wet-weather flow information is needed. However, the primary emphasis is on dry-weather flow. Dry-weather flow records will identify the base flow from infiltration and inflows (basement sump discharges, cooling water, surface drainage, etc.) that limits the capacity of the storm sewer for conveyance of the storm flows. It is also necessary to determine the capacity available in sanitary sewers if stored flows are to be bled into the sanitary sewer for conveyance to an existing treatment plant.

Drainage Area Characteristics. The characteristics previously identified for combined sewer systems also apply to separate storm sewer systems. These include topography, land use, geology and soils, and size.

Suspended Solids Characterization. The need for suspended solids characterization of the stormwater in separate storm sewers is similar to that for combined sewer overflows. The procedure and application should be the same as described previously for combined sewage.

Collection System. The configuration of the collection system will determine where storage or sedimentation facilities can be located. In-system storage is best accommodated where pipes are large and relatively flat. Inline and offline storage can be located throughout the collection system.

Discharge and regulator locations can be used to control flow to storage or sedimentation facilities. Additional controls may be required to direct flow into or discharge flow from facilities at these locations.

Existing pumping stations can be used to effect storage within existing storm sewers by adjusting the pump operation controls. Pumping stations can be used to direct selected flow to a storage facility or direct all flow to a sedimentation facility. Discharge from a storage facility can be controlled by a pumping station also.

Determine Pollution Load

The need to determine the pollution load for separate storm sewer systems is the same as for combined sewer systems as described previously.

Since the particle size distribution and the particle volume distribution do not remain constant during the runoff process, it is necessary to determine the typical change of these factors with time. This is true not only for suspended solids but also for any other pollutants of interest that are associated with the suspended solids. The change of pollutant load with time will determine whether storage of a certain volume of runoff or sedimentation for the entire runoff will be most effective in removing the desired pollutants. If several pollutants are of concern, a combination of storage and sedimentation may be necessary depending on the pollutant load variation with time.

Identify Flood Control and Pollutant Removal Objectives

At this point, it is necessary to identify the flood control or pollutant removal objectives or both to be used for design. Storage or sedimentation facilities can be used not only for pollutant removal but also for flood control.

Flood Control Only. If flood control is the only objective, storage would be used rather than sedimentation. The storage could be onsite, in-system, inline, offline, downstream, or any combination thereof. The facilities would be designed strictly for flood control and the pollutant removal that occurs when storage is used would be simply accepted.

Dual Purpose. When the pollutant loads are known, the decision can be made to incorporate pollutant removal as a stragegy into the control facilities. In this case, the design of the facilities is such that both flow control and sedimentation are included. However, the two are not necessarily weighted equally. The overriding design concern may be flood control, but with the maximum sedimentation possible consistent with the flood control need.

If a dual need is identified, it is also possible to retrofit existing storage of flood control facilities to improve pollutant removal. Innovative designs may be necessary to effect improved pollutant removal in existing flood control facilities.

New facilities can be designed while maximizing both the flood control and pollutant removal objectives.

Control Optimization

The objective of any separate storm sewer control system should be to maximize the effectiveness of the control system by producing the greatest desired effect for the funds available. The desired effect is the objective(s) identified in the previous section. The steps involved are essentially the same as those described in the Combined Sewer section.

Identify Sources of Storage. The storage can be centralized or dispersed throughout the separate storm sewer system. It can be onsite before it ever enters the sewer system, in-system as online or offline storage, or downstream just before discharge to the receiving water.

Identify Treatment Locations. Treatment locations of the pollutants removed in storage or sedimentation facilities must be identified. The sediment removed from the storm flow can be discharged to the sanitary sewer for conveyance to existing dry-weather water pollution control plants for treatment and disposal. Another option is to have a dual-use treatment plant that handles both dry- and wet-weather flow. Such dual-use facilities are usually new treatment facilities or expansions of existing dry-weather treatment facilities.

Cost Analysis. The cost analysis procedure is the same as described for the combined sewer overflow control.

Optimization. The effect on the cost and pollutant removals of the variations in the size and location of the various facilities should be determined. For the funds available, the locations and sizes of the required facilities is optimized to provide the greatest effectiveness.

Pollutant Budget Analysis

To determine the overall effectiveness of the flood control and sediment removal facilities, it is necessary to perform a pollutant budget analysis. A mass balance of the residuals (solids) in storage during both dry and wet periods is needed. This is necessary so that the pollutant loadings on both the treatment facilities and the receiving water can be determined. The treatment facilities must be able to handle the solids during an after-the-storm event. Those solids include any deposited by dry-weather flow in the storm sewer as well as those from the storm flows.

If an objective is to minimize the pollutant loading on the receiving water, any sediment removal facilities must not have accumulations of dry-weather pollutants that could mix with the storm flow to produce a greater pollutant concentration in the outflow from the storage or sedimentation facility than would have occurred if the stormwater were allowed to pass untreated.

The pollutant budget analysis may act as a modifier to the flood control design if pollutant removal is a major objective. In other words, the design of the facility for pollutant removal may take precedent over the flood control design criteria.

Operating Strategy for Design

It is necessary to determine an operating strategy for the design of the separate storm sewer control facilities. As with the combined sewer overflow control facilities, the alternatives for the physical layout of the facilities must be selected (both locations and sizes), the pollutant budget analyzed and refined, and an optimization analysis performed. If necessary, this may be an iterative process to determine the apparent best alternative for implementation.

RETROFITTING OF EXISTING FLOOD CONTROL FACILITIES

Temporary storage is one of the most commonly applied flood control techniques. Many existing flood control detention facilities may be modified to enhance pollution control of the stormwater as well.

The most common flood control facilities in the United States are wet and dry ponds [23]. In many cases, the pond is not designed to maximize the potential for pollutant removal while acting as a flood control facility. However, often it is possible to retrofit such facilities so that they serve as both flood control and pollutant removal facilities. Toward that end it is important that the detention time be maximized; horizontal velocity gradients, vertical velocity gradients, and turbulence near the inlet and outlet be minimized; and access be provided for removal of sediment and floatable debris. Pollutant removal in a flood control pond or facility is most often accomplished by sedimentation; however, biological action and adsorption in the soil resulting from percolation and infiltration can also remove pollutants.

To promote effective sedimentation in a flood control facility, the following design features should be incorporated [2]:

1. Length:width ratio greater than 2:1 where the length is the straight line distance between inflow and outflow.

2. Wedge shape with the inlet at the apex of narrow end.

3. Minimum length between inlet and outlet of 3 ft (1 m) for each acre of watershed.

4. A surface withdrawal system to minimize resuspension of deposited materials.

5. Minimum permanent pool depth of 3 ft (1 m) for wet ponds.

6. Provision for complete drainage.

For dry ponds, a swale should be provided in the pond bottom so that any dry-weather flow can be transported directly to the outlet. The outlet should be at the invert of the swale and be designed to pass the dry-weather flow unrestricted.

Several modifications are possible to increase the pollutant removal efficiency of existing or planned flood control facilities. Gravel or cement may be added to the pond walls and bottom to prevent erosion by stabilizing the soil. Baffles or other energy dissipation devices installed near the pond inlet may distribute flow and reduce turbulence caused by the velocity of the flow entering the pond. The pollutant removal performance of wet ponds can be ensured or improved by keeping the length:width ratio greater than 3:1. Installation of vertical baffles, similar to a fence that extends from the pond bottom to the high water level, can provide a series of channels that guide the flow on a route that promotes plug flow (first in, first out) through the pond. Such channels should have a width:depth ratio of from 1:1 to 2:1. This helps prevent short circuiting of the flow and excessive suspended solids escape through the outlet caused by insufficient detention time. Concrete lining of the area near the inlet where the heavy sediment settles makes removal of that sediment much easier.

Since biological activity, straining, and adsorption take place as flow infiltrates and percolates through the soil underlying a pond, increasing the infiltration capacity will improve the pollutant removal efficiency. Installation of underdrains in dry ponds is one method of increasing infiltration capacity. The pollutant removal performance of wet ponds may be increased by raising the level of the outlet pipe. The permanent pond size will be increased and sedimentation performance should improve. Of course, some flood control capacity will be lost.

Some pollutants, particularly nutrients, are utilized by plants growing in the pond. If the plants are allowed to die and decompose, the nutrients are released back to the pond. Harvesting and proper disposal of such vegetation will remove those pollutants from the stormwater control system [24].

INTEGRATION PROCESS EXAMPLES

The general principles presented in the integration section can be practically explained by use of illustrations. In the first example, a storage and/or sedimentation basin is to be placed in a developed urban area to mitigate flooding and stormwater pollution impacts. The process of choosing a location for the basin and examination of the applicability and compatibility of this control method are examined.

The second example involves the retrofitting of a flood control facility to improve the pollutant removal effectiveness and improve the quality of the stormwater discharged. The third example describes the integration of a retention and attenuation facility into a developing area.

Storage and/or Sedimentation Basin Integration (Flood and Pollution Control)

Assumptions. The siting of a storage and/or sedimentation basin involves three factors: existing facility interface, land use compatibility, and space availability. The preliminary phases of integration planning and data gathering had identified a storage and/or sedimentation facility as the most appropriate control method for accommodating the stormwater from a portion of the city's separate storm sewers.

The goals of stormwater management are: (1) to eliminate flooding along a river, and (2) to reduce the pollutants in overflows to a river. The storage and/or sedimentation facility was selected as a control method because storage volume is required to prevent flooding in the urbanized portion of the city and sedimentation also afforded a pollutant reduction mechanism to improve overflow quality.

For the basin to interface with existing facilities, it must be located near existing sewers. It should also be sufficiently close to the treatment plant for minimizing sludge transmission distance and upstream of the area with storm sewers subject to flooding during high runoff periods. The areas meeting these criteria are in an area of the city where the land uses are commercial and light to medium industry. Within the area are a few unimproved lots used for parking and some older warehouses where renovation is planned. The installation of a storage and/or sedimentation basin in the area would be compatible with existing land uses.

Due to limited space availability, the locations for the basin are narrowed to two potential sites: an existing parking lot and a warehouse scheduled for demolition and renovation. The basin, if located at the parking site, could be constructed to allow parking above it, thus permitting dual use of the site. Alternatively, the warehouse site is proposed as a recreational facility for workers in the area, which also offers a dual use potential.

Functional Compatibility. In initial planning stages, the functional compatibility between the basin and existing facilities is briefly considered only in selecting a general area to search for compatible locations. During the integration process, the flow scheme, impacts, and operation and maintenance aspects must be investigated in more detail for functional compatibility.

The proposed flow scheme is to have the basin offline and connected to the storm sewer by regulators. The primary function of the basin is to prevent flooding. When flow in the sewer reaches maximum stage without flooding, the basin is to be brought online to intercept and hold the stormwater excess. When the peak flow subsides, stored water will then be drained back into the sewer.

The impacts of the storage/sedimentation basin in the storm sewer system are to reduce peak flows and remove solids. Should wet-weather treatment become necessary in the future, the basin will help to distribute the flow volume over a longer period enabling a smaller maximum design capacity for the treatment plant while maximizing the volume of the flow receiving treatment.

The operation of the basin will be triggered automatically by flow depth sensors in the sewer. Sludge is discharged to the sanitary sewer following the storm. Drainage of the basin under normal operation is to the storm sewer. Maintenance and cleanup operations will occur after each storm.

An alternate drainage path for the facility is through the dry-weather treatment facility. Current design of the basin has the dry-weather drainage path option, as well as permitting a potential reverse flow path from the dry-weather facility to the basin. The pipe, to allow dry-weather flow diversion to the basin, is to be installed at the same time the basin sludge discharge line is being constructed.

Process Compatibility. The two flow schemes for the basin have different impacts on the treatment facilities. Flexibility, capacity, and quality impacts are discussed. The flexibility of the system is high. The basin is designed to operate as a peak flow detention facility with the added ability of being able to route flow to the dry-weather treatment facility. For dry-weather plant failure or emergency conditions, the basin could provide standby storage for subsequent return to the dry-weather plant for treatment.

Additional flexibility is obtained by using the connection from the dry-weather plant to the offline storage for temporary storage of dry-weather flows during peak periods. When the treatment plant reaches its design hydraulic capacity, the useful function of the plant is extended by storage of peak dry-weather volume and then processing it during low flow periods. This flexibility can only be achieved during non-storm periods. In all of the operation options, odor control measures must be implemented; therefore, the dry-weather processing option in this case does not impose added design cost due to odor control.

The impacts of the basin on treatment facilities are both positive and negative. The positive impact includes postponing capacity expansion of the dry-weather plant due to mitigating peak flow demands. The negative impact includes increased maintenance and operation of the sludge processing facilities. If the sludge process requires degritted sludge, then arrangements for degritting the wet-weather sludge must be provided. Options include installation of grit removal equipment at the basin or introduction of the sludge ahead of grit removal equipment in the dry-weather treatment facility. Since the stormwater flows are seasonal and sporadic, the demands which stormwater processing imposes on treatment facilities are highly variable. Thus, the treatment plant should be able to handle the additional loads since they will not drastically increase the average annual load on the plant.

The development of treatment facilities for the city is still undergoing analysis. To meet flow and quality requirements, the city will have to construct treatment facilities. Available options are expansion of the dry-weather plant to accommodate some stormwater treatment, or construction of new separate facilities for wet-weather processing. The storage and/or sedimentation basin is an integral part of either option. The functioning of the basin in regard to dry-weather facilities was previously discussed. The basin will operate in a similar fashion with wet-weather facilities. The basin's primary function is to prevent flooding and, by storing flow, permit a reduced peak design volume for the downstream treatment plant. Sludge would be transferred to expanded sludge facilities at the dry-weather site.

The improvement in quality of the discharge to the river will consist primarily of reduced sediment loadings. The load of heavy metals and other pollutants associated with sediment will also be reduced in the discharge. The number of overflows from the area served by the basin will be reduced, with a corresponding increase in discharge quality due to processing through the treatment plant.

Flood Control Retrofit

Assumptions. To alleviate the flooding in a portion of the city, a flood control basin was constructed on city property adjacent to the river. The embankment around the basin and along the river was raised to prevent flooding. Stormwater is discharged to the river from the basin by both pumping and gravity. Flap gates were installed on the gravity discharge pipes to prevent river water from entering the basin. When the water level is high in the river, stormwater is discharged from the basin by pumping. When the water level in the river subsides, stored runoff flows to the river by gravity.

With the increased concern and regulations regarding stormwater pollutant discharges, the city has determined it would be cost effective to retrofit the existing flood control basin to improve the quality of the stormwater discharge into the river.

Facility Modifications. The modifications needed include physical and operational changes. The current mode of operation is to drain runoff into the river as quickly as possible. When gravity flow into the river is possible, no buildup of runoff volume occurs in the basin. When pumping is required, the volume in the basin is allowed to accumulate sufficiently to prevent frequent, short-period operation of the pumps. Most of the volume, however, is kept in reserve for sudden peak runoff occurrences.

The operational changes required involve detaining a volume of stormwater sufficiently for sedimentation to occur. During low river level, gravity flow would fill the basin, overtop a weir (or other restraining device), and continue by gravity flow to the river. During the pumping phase, effluent from the sedimentation basin would flow to the pump sump. For peak flush periods, a partial flow bypass from the entry of the facility to the sump would prevent velocities in the sedimentation basin from getting high enough to resuspend the sediment. However, a final decision on the inclusion of a partial flow bypass should not be made until a pollution budget analysis has been completed. The pollution budget analysis will help to determine whether the most efficient removal at high flows occurs when part of the flow is bypassed or when all of the high flow is passed through the basin.

The physical modifications to permit the above operation mode include installation of stilling basin, weir (or other restraining device), sediment removal equipment, compartmentalization of the basin, and peak flush conveyance channel.

To create the sedimentation basin, a weir or a stop log type of flow restraint is installed. The flow then is subject to detention time in the basin. The flow then proceeds from the basin to the pump sump.

By compartmentalizing the basin, only a portion of the basin is used for sedimentation while the remainder is available to handle the peak flow without sedimentation. This prevents sudden large flows from resuspending material in the sedimentation basin. An increase in the pumping capacity is necessary to offset the loss in standby volume in the sedimentation portion of the basin.

To keep the basin operating as desired, sediment must be removed on a routine basis. The frequency of the required sediment removal operations is dependent upon the volume and configuration of the basin, the frequency of storm events, and the suspended solids loads associated with the storm events. For small basins, sediment may have to be removed following each storm event or after only a few events. Sediment may be allowed to accumulate for several years between cleanings in very large basins.

Installation of sediment removal equipment in small basins that must be cleaned frequently may be required as part of the retrofitting of the facility. For large basins where long periods between cleanings are acceptable, modifications may include provision for access for cleaning and maintenance work.

Functional Compatibility. The flood control facility, as retrofitted, is compatible with stormwater management goals. Impacts of the retrofit of the facility on flow handling are minor. The decrease of the storage volume is offset by increased pumping capacity so a higher volume of stormwater will be pumped at the design storm peak flow, but there is no practical change in the ability of the system to protect the residential area from flooding.

Operation and maintenance of the facility is altered due to increased cleanup activities after storms and higher maintenance requirements. The facility is self-actuating so the beginning of a storm event does not necessitate rapid mobilization of staff.

Process Compatibility. The quality impacts of retrofitting the flood control facility are to reduce the amount of sediment and associated pollutants released during storm events. Except for the largest storms, most of the storm flow undergoes primary treatment. During bypass conditions, only part of the flow is subject to primary treatment. If the first flush phenomenon applies to the city's stormwater system, the retrofit flood control facility will detain the first flush volumes for treatment, enabling a greater percent removal efficiency on an average annual basis.

Retention and Attenuation Facility Integration

Assumptions. An existing cattle ranch has been rezoned for single family residential use with a local commercial business district. A small creek channel, with a pond for watering cattle, crosses the property and enters the storm sewer in a presently developed area of the city. The creek is dry except during the spring. Storm and sanitary sewers will be extended into the developing area. The city has an ordinance requiring the peak runoff rate from any future development not exceed the peak rate from the undeveloped site.

The potential retention and attenuation control methods for this site are numerous. Rooftop storage, parking lot ponding, and low structural need retention or detention basins offer the highest potential for being optimum stormwater management control techniques. The stormwater control measures can be easily integrated as the site plans are being developed.

The adaptability of the control methods for implementing storage or sedimentation or both and retention is an advantage. As new development in the area continues, the most appropriate control method can be chosen for each subsequent site without conflicting with other areas. For an expanding system, the net effect is that each expanding area adds facilities as required, which prevents negative impacts on downstream facilities.

Control methods need to be selected with compatibility in mind. Rooftop storage, plaza ponding, and parking lot ponding are all compatible with commercial business land use. Retention or detention ponds can be compatible control methods with residential land uses. Other facilities requiring more structural components for storage or sedimentation or both also can be compatible. Low structural need control facilities can be blended with the natural surroundings.

The low structural need facilities often require larger land areas. In the new development, there is sufficient land for retention ponds. The options are open to have dry ponds or wet ponds. The cattle watering pond, having been in existence for many years, has developed natural vegetation for pond areas. This portion of the site could be converted to a wet pond with additional storage volume requirements being provided by dry pond storage.

Functional Compatibility. After analysis of the site to optimize the integration of stormwater control methods, the following combination of controls was selected. For the commercial business area controls include: (1) rooftop storage and controlled rate of release, and (2) parking lot storage with drains to a small structural basin to aid in petroleum hydrocarbon and sediment removal and to add additional required volume. The additional volume was required because, without becoming a nuisance or hazard, parking lot storage was not sufficient to handle the business district runoff volume. The designed rooftop and parking lot storage is expected to be sufficient to detain the peak flows and prevent exceeding the storm sewer capacity.

The residential section has both a wet and dry pond to regulate storm flows. The wet pond was established at the cattle watering pond. A dike surrounding the pond was built up sufficiently to provide for storage above the normal level of the pond. The wet pond serves as a retention basin for most storms. The large storm runoffs exceeding the capacity of the wet pond continue on to the dry pond. The dry pond also was established as a recreational field during periods without rain. Drainage swales throughout the development have stepped barriers to add to the detention storage volume available.

The required maintenance procedures can be distributed over a period of time and would not, therefore, represent a labor intensive period at any point during the year. Channels must be kept free of debris. The rooftop units must be checked periodically. The parking lot area should be kept swept with normal street cleaning procedures. The dry pond must be maintained in grass, free from excess vegetative growth. Plant growth around the edges of the wet pond also must be controlled.

Additional maintenance around the wet pond is required after storms to remove accumulated floatable materials. Dredging or scraping the bottom of the pond periodically will prevent loss of recreational use due to sedimentation. None of the control methods conflict with the operation of the storm sewer.

Process Compatibility. Retention and attenuation facilities are compatible with any of the other treatment processes. Since the control measures in the new development are primarily flow control measures, little impact of treatment processes will be observed. The only quality effects will be from sweeping of the parking lot to reduce the quantity of pollutants in the runoff and the wet retention pond that will contain pollutants from part of the residential section. The chosen control methods, being independent of the other stormwater management controls, do not impose constraints on downstream facilities and therefore allow for maximum flexibility.

Treatment capacity will not be negatively impacted, since the new development is designed to have the same net runoff effect as the unimproved property. A slight decrease in pollutants is expected, causing a reduction in pollutant loading. The accumulated sediment removed from the wet and dry ponds must be disposed of at appropriate landfill sites.

The quality impacts of this stormwater management system are to slightly reduce pollutant loadings in the stormwater runoff reaching the river. Groundwater impacts of retention facilities must also be examined. Stormwater runoff from developed areas may not contain high sediment loads but additional pollutants such as petroleum hydrocarbons, heavy metals, and other toxic compounds are higher than from the undeveloped area. This should be taken into consideration when percolation is used for disposal of stormwater.

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SECTION 5

DESIGN OF RETENTION STORAGE FACILITIES

Stormwater retention is the storage of excess runoff for complete removal from the surface drainage and discharge system. Stormwater retention facilities may take a variety of forms. For instance, in Orlando, Florida, perforated aluminum culverts have recently been installed below street level in the downtown area to store and percolate ground water [1]. This section describes design procedures and operation considerations for the most common retention storage facility type--the pond. Stormwater retention ponds may be divided into two general categories: dry ponds and wet ponds. Dry ponds are earthen basins that are wet only during and immediately after runoff events. Excess flows are directed to them and allowed to percolate to the groundwater. Wet ponds are permanent ponds in which stormwater is stored by varying the level of the pond. Stormwater also percolates to the groundwater from wet ponds.

Early in 1980, the American Public Works Association conducted a survey of stormwater storage practices in the United States and Canada [2]. Of the 12,683 facilities reported, almost 50% were dry ponds. An additional 2,382 facilities were wet ponds. Not all, however, operate as retention basins.

Percolation of stormwater to the groundwater offers a number of benefits in addition to controlling stormwater flows. The groundwater is recharged. A total of 1,513 facilities were reported in use for groundwater recharge. This is particularly important in areas where the groundwater basins are being overdrawn and increased urbanization is reducing normal infiltration. In addition, percolation through a soil column has been shown to be very effective in removing bacteria, oxygen demanding material, and suspended material from a wastewater. However, dissolved toxic materials may pass through to the groundwater.

DESIGN CONSIDERATIONS

Runoff storage and percolation are the primary ways in which dry ponds reduce pollutant loadings to receiving waters. As with other stormwater storage facilities, ponds allow sedimentation removal of suspended materials during overflows. Additional pollutant removal in wet ponds may also result from biological oxidation of suspended and dissolved organic material in the runoff. Soil characteristics and permeabilities play an important role in design and operation of these facilities. In addition, ponds frequently are designed to serve multiple purposes, usually for flood control and recreational facilities. Other purposes also include aesthetics as well as, less frequently, stormwater pollution control facilities. Dry ponds may serve as playgrounds or athletic fields when not in use for stormwater control while wet ponds are often also recreational lakes. Other uses (storage reservoir, drainage control, and improved local aesthetics) may have as great or greater impact on size, location, and configuration decisions as pollution control. This section includes a discussion of design considerations and factors for both types of ponds.

Size

Size requirements include not only volumetric capacity but also both surface and soil interface area requirements, as well. The pond configuration depends on:

- The runoff storage volume needed.
- The surface area and weir length required to assure adequate settling during sedimentation operation.
- The surface area needed for adequate transfer of oxygen into the pond water to allow aerobic decomposition of organic pollutants.
- The soil-water interface area needed for adequate percolation of stored runoff between storm events.
- The area needed to serve whatever dual uses the basin may have.

The ideal pollution control design is a balance of storage capacity and sedimentation removal that will yield the necessary wasteload reduction for the lowest cost. Sizing determinations, based on these factors, are the same as for detention storage/sedimentation basins discussed in Section 7.

The storage volume required is also a function of soil characteristics of the pond site, particularly soil permeability, subsurface geologic conditions, stormwater pollutant makeup, and the antecedent dry-weather period between runoff events.

Permeability is a term used to describe the ease with which liquids and gases pass through soil. In general, water moves through soils or porous media in accordance with Darcy's law (Figure 16):

$$q = k\ dH/dl \qquad (5\text{-}1)$$

where q = the flux (rate of flow of water per unit cross-sectioned area), in./h (cm/h)

k = the permeability, in./h (cm/h)

dH/dl = the toal head (hydraulic) gradient, ft/ft (m/m)

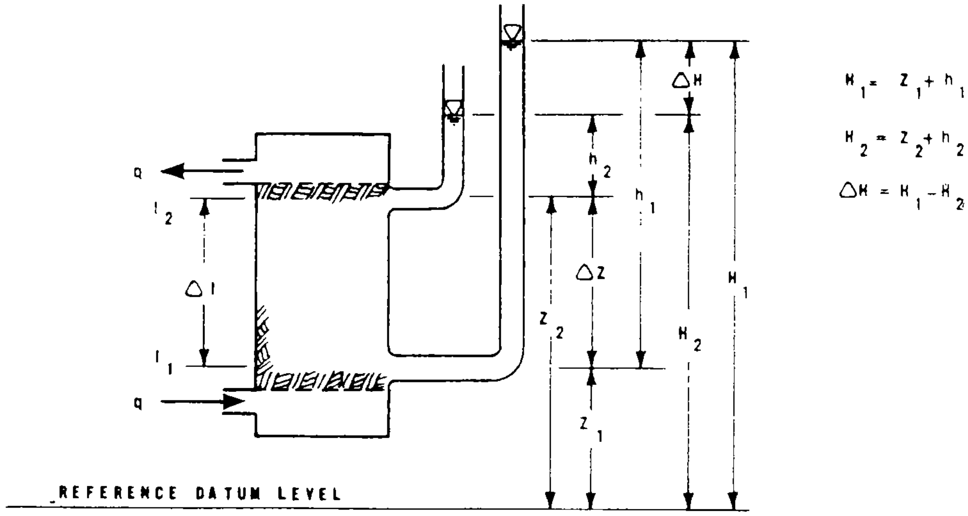


Figure 16. Schematic showing relationship of total head (H), pressure head (h), and gravitation head (Z) for saturation flow [3].

The total head (H) is the sum of the soilwater pressure head (h), and the head due to gravity (z), or H = h + z. The hydraulic gradient is the change in total head (dH) over the path length (dl). The permeability is defined as the proportionality constant, k.

Soil permeability is determined to a large extent by soil texture with coarse materials generally having higher permeabilities. In some cases, the soil structure may be of equal importance. A well-structured clay with good stability, for example, can have a greater permeability than a much coarser soil. The ionic nature of the soilwater (capillary or contact water that remains in the soil after groundwater has drained by gravity) and type of vegetation may also affect permeability by reducing the effective pore size in the soil.

Percolation, the movement of water through the soil, is a distinctly different property from infiltration. The infiltration rate of a soil is defined as the rate at which water enters the soil from the surface. When the soil profile is saturated, the infiltration rate is equal to the effective saturated permeability of the soil profile. When the soil profile is relatively dry, the infiltration rate is higher because water is entering large pores and cracks. When water is applied, large pores fill and clay particles swell, reducing the infiltration rate to a near steady state value.

As with permeability, infiltration rates are affected by the ionic composition of the soilwater and by the type of vegetation. Of course, any tillage of the soil surface will affect infiltration. Factors that tend to reduce infiltration rates include clogging by solids in the applied water, gradation of fine soil particles, clogging due to biological growth, and gases produced by soil microbes.

Field measurements of soil infiltration rates and permeabilities are an essential part of the design of retention/percolation basins. Generally, the steady state infiltration rate serves as the basis of selection of the design hydraulic loading rate. The rapidity with which water moves through a soil profile will be determined by the least permeable layer in the profile. However, subsurface permeabilities must also be considered.

There are many available techniques for measuring infiltration, including flooding basins, sprinkler infiltrometers, cylinder infiltrometers, and lysimeters. The preferred technique is one that approximates the actual method of water application. In the case of dry ponds, this would be flooding basins or cylinder infiltrometers. It is strongly recommended that hydraulic tests of any type be conducted with actual stormwater when possible. The infiltration rate of a particular site will tend to decrease with time as stormwater is applied as a result of continuous inundation which excludes oxygen from the soil, thus engendering the growth of anaerobic bacteria on the organic matter deposited on or contained in the soil with resultant clogging of the soil system [4]. This clogging generally occurs only at the surface, and the infiltration rate may be returned to nearly its original value by scarifying the surface or permitting it to drain and to reestablish aerobic conditions [4]. Selection of a design infiltration rate must take into account this clogging [3].

Particularly for wet ponds, the biodegradable organic content of the runoff may also influence pond sizing. Biological stabilization of organic materials is accomplished with the use of dissolved oxygen, if available. If the dissolved oxygen is depleted, anaerobic decomposition occurs, producing odorous gases and discoloring the water. Oxygen is dissolved into the water at the air-water interface or from oxygen released by algae in the water. The rate at which oxygen is used is a function of the organic loading and of the water temperature. The rate at which oxygen is dissolved depends on the magnitude of the oxygen deficit and the surface turbulence. Therefore, organic loading per unit surface area may be an important consideration. The acceptable loading rate depends on alternative pond uses. Domestic wastewater treatment ponds are usually loaded at 15 to 35 lb BOD_5/acre·d (17 to 40 kg/ha·d) [5]. It is recommended that stormwater retention ponds not be loaded at higher than 5 to 10 lb BOD_5/acre·d (6 to 11 kg/ha·d). An active biomass to provide BOD reduction in domestic wastewater treatment ponds can be maintained since the flow and pollutant loadings remain relatively constant; the intermittent flow and variable pollutant loads reaching stormwater retention ponds are not conducive to maintaining a stable active biomass. Organic loading usually is not a problem for ponds that control stormwater runoff. Anaerobic conditions may result in ponds with small surface areas used to control combined sewer overflows.

To avoid the generation of malodorous gases or the development of nuisance insect populations, it is recommended that dry ponds should be designed to allow complete percolation of the full pond retained flow in not more than 7 days for stormwater runoff, or 3 days for combined sewer overflows.

In most cases, flood control is a dual purpose of stormwater retention/percolation ponds, and the flood control and hydrograph attenuation needs usually determine the storage volume required. The 1980 APWA survey revealed that for detention facilities, the most frequently cited basis for flood control storage sizing is the 100 year rainstorm, followed by the 10 year and the 25 year storm, in that order [2]. Design of ponds specifically for control of flooding is not within the scope of this manual, but is adequately discussed in the literature.

Location

The suitability of various sites within a drainage area for pond facilities depends on (1) the availability of the site, (2) compatability of surrounding land uses with a stormwater retention facility use and other dual use functions, (3) the area required, (4) the soil characteristics, and (5) the location of the site with respect to tributary catchment size and to other sewer or drainage facilities.

Evaporation may also be a factor in the disposition of water from retention/percolation ponds. For dry ponds, its effect is slight, since such ponds are usually designed to empty within 7 days, and evaporative losses over such a short period are small. Evaporation may be a consideration in maintaining the permanent pool in a wet pond, particularly in areas subject to seasonal rainfall. In the western United States, for instance, a retention/percolation pond may operate as a wet pond during the winter rainy season and be dry during the summer.

The first consideration in identifying potential locations for a stormwater control facility is the availability of a given site. For new development areas where urbanization is yet to occur, this is not usually a problem. For installations in established areas, site availability can be an important factor. Ideal locations may already be occupied by buildings or highways. Even if the agency developing the control facility has powers of eminent domain, exercise of such powers should be a last resort. In any case, fair market value of an already developed site may make it prohibitively expensive. Any site which has already been developed should be considered for redevelopment as a stormwater control facility only after all undeveloped sites have been identified, evaluated, and rejected.

Another important consideration for retention basins is the compatability of land uses in the surrounding area. For facilities in or near residential or commercial areas, more intense operation and maintenance efforts are required to prevent nuisance conditions from occurring. Less stringent operation and maintenance efforts may be required for retention facilities in more remote or less visible locations.

Obviously, the size of the facility needed and the site soil characteristics play very important roles. Preliminary screening of sites may be accomplished based on information from soil maps. Final designs must be based on field testing of soil permeabilities.

First flushes, which often occur in both storm sewer discharges and combined sewer overflows, usually are of a shorter duration for small tributary catchments than for larger catchments. Thus, since the volume associated with the first flush from a small catchment would be proportionately less than that from a large catchment, several small retention basins may be more appropriate than a single large basin. If first flush control is the established goal, retention/percolation basins should be located accordingly. In addition, rapid percolation of a stored volume of stormwater runoff, so that storage volume is available in time for the next storm, usually requires a large soil-water interface area. If the catchment is large, the necessary pond area will also be large.

Location of the site with respect to the drainage and/or sewer system is another factor. Ideally, locations should be selected to minimize transport drainage conduits/channels from the existing drainage facilities, and also to allow discharge of basin overflows with minimum of outfall piping construction.

DESIGN PROCEDURE

The following section consists of a step-by-step procedure for design of retention ponds. The first two steps are typically carried out at a planning stage, and are discussed only briefly. The approach used in the design of retention facilities should make use of existing experience, known concepts, and developing theories. An integrated design procedure must be used to insure that the desired functions of the pond (sediment removal, infiltration and percolation, flood control, or flow reduction) are compatible with the types of flow reaching the pond (stormwater runoff or combined sewer overflow) and any other multi-use aspects (recreation, aesthetics, etc.). In actual practice, retention ponds are very seldom used for combined sewer overflows because the organic solids tend to seal the pond bottom and reduce the infiltration capacity.

Step 1 - Quantify Functional Requirements

Using an accepted hydrologic analysis method, determine storm distribution patterns for the urban area being analyzed. This should include a statistical distribution of storm volumes, storm intensities, and storm durations and frequencies. Historical rainfall data are available from a variety of sources. The length of the rainfall time-step used to calculate the runoff depends on the size of the watershed being analyzed, the length and intensity of the storm, and the degree of accuracy desired. Typically, analysis of a small watershed may require a 5- to 10-minute interval while a large watershed may only require a 1-hr interval. Hourly rainfall data for many locations in the United States is available from the National Weather Service. Shorter interval rainfall data are often available from drainage, sewerage, flood control, or other special districts.

Runoff is related to rainfall occurrence. Many methods are available; often, a regional flood control agency will specify the runoff calculation method to be used. For a first-cut analysis, a single event hydrograph may be sufficient. However, for the final design a long-term hydrological evaluation should be used and analyzed in the same fashion as a long term streamflow record.

Next, pollutant characteristics should be determined. Actual field data specific to the catchment being considered or to a nearby and similar catchment should be collected. Included should be pollutant types and concentrations. Particle size distributions, specific gravities, and/or settleabilities or settling velocities of suspended solids and associated pollutants of concern should be determined. Information on settleability characteristics is discussed in Section 7.

If a significant first flush of pollutants or suspended solids is evident, the time variation of the mass loading rate can be an important consideration in the design of the retention facilities. If most of the pollutant mass load occurs during the early part of the storm event, it is important that that portion of the flow be retained while later, more dilute portions of the flow are allowed to pass on downstream. Significant solids depositions could also affect operation and maintenance of the retention basins and need to be considered in planning and designing the basins.

Step 2 - Identify Required Waste Load and Flow Reduction

In some cases, regulatory agencies may specify the level of flood control required; the level of pollution control may also be specified but only in a few cases. If an allowable pollutant load is not specified, the expected impacts of the stormwater on the established beneficial uses of the receiving water may be used to calculate the pollutant waste load reductions required.

Step 3 - Determine Preliminary Basin Sizing

Since a dual purpose of most stormwater retention ponds is flood control, preliminary determination of storage volume needs is often based on flood control requirements. Flood control requirements are usually expressed as control of runoff peak flow from a given design storm to some specified rate, often the predevelopment rate. The effect of a storage pond on runoff peak flows is estimated by a flow routing procedure.

Based on the flood control aspects alone, the retention basin can be treated as a simple reservoir. Flow routing for a reservoir requires that three relationships for the reservoir be known: (1) an inflow hydrograph, (2) a depth-storage relationship, and (3) a depth-discharge relationship. Routing is the solution of the storage equation which is an expression of continuity:

$$\bar{I} - \bar{O} = \Delta S/\Delta t \qquad (5\text{-}2)$$

where I = inflow rate
O = outflow rate
S = storage
t = time

Using subscripts 1 and 2 to represent the beginning end of the period, respectively,

$$\frac{I_1 + I_2}{2} - \frac{O_1 + O_2}{2} = \frac{S_2 - S_1}{t_2 - t_1} \qquad (5\text{-}3)$$

Equation (5-3) may be transformed to

$$I_1 + I_2 + \left(\frac{2S_1}{t}\right) - O_1 = \left(\frac{2S_2}{t}\right) + O_2 \qquad (5\text{-}4)$$

where t = time (routing period)

Solution of equation (5-4) requires a routing curve showing $2S/t + O$ vs. O. All terms on the left-hand side of the equation are known and a value of $2S_2/t + O_2$ can be computed. The corresponding value of O_2 can be determined from the routing curve. The computation is then repeated for subsequent routing periods.

The depth-discharge relationship can be a composite made up of the relationships for multiple outlets. An example of the routing curves for typical reservoir is shown in Figure 17.

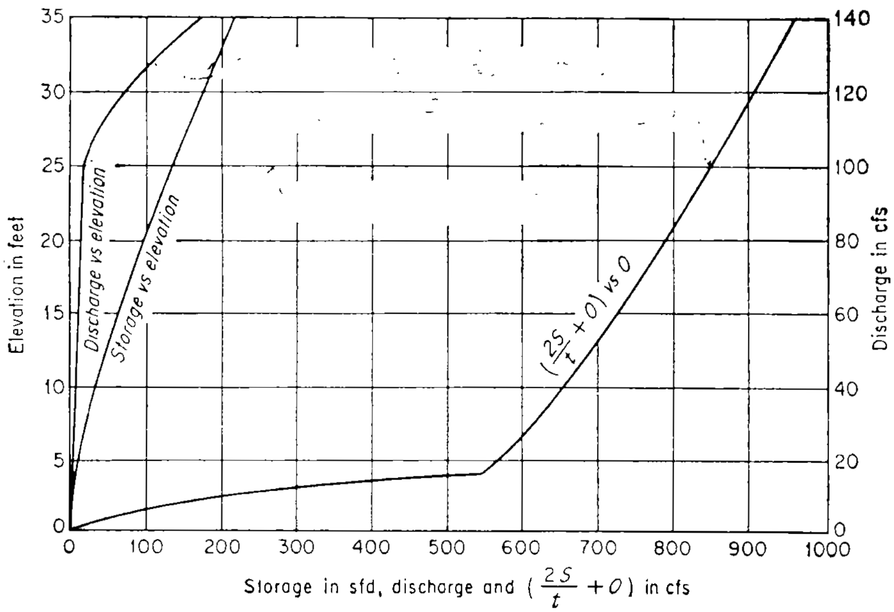


Figure 17. Routing curves for a typical reservoir.

Alternative flood routing procedures, either hand computation, computer based simulations, or graphical methods, may also be used.

The effectiveness of the pond in removing heavier sediments (soil particles) can be estimated using the curve shown in Figure 18, based on Brune's work [6]. Estimating removal efficiencies for lighter materials, such as organic solids, is not so easy, since the effects of eddies and currents within the basin are more pronounced for such particles.

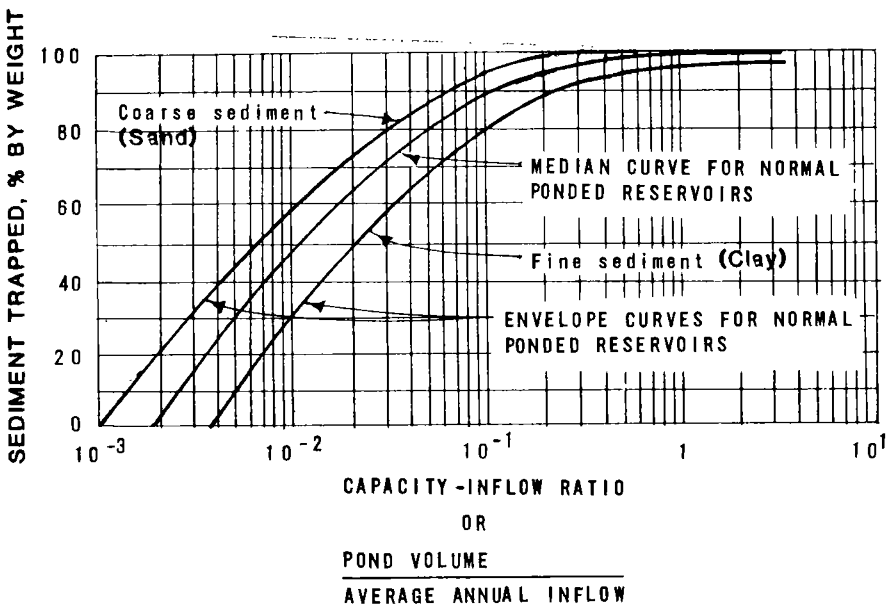


Figure 18. Brune's trap efficiency curves [6].

A more rigorous method for estimating pollutant removals is the utilization of a mathematical model for either single event or continuous simulation. The two methods used in the Storage/Treatment block of SWMM-Version III are examples [7]. In the first case where pollutants are characterized only by mass flow and concentration, the removal may be simulated as a function of detention time, influent concentration, inflow rate, removal fraction of another pollutant, incoming concentration of another pollutant, or any combination of the above. The selection of the equation variable is left to the user. In the other case, pollutants are characterized by their particle size and specific gravity distribution, (or apparent specific gravity distribution depending on the amount of air or oil in the interticies of the particles), then are removed by particle settling. The removal is determined by summing the effects of several ranges passing through the unit. Application of these methods provides improved estimates of the removal efficiency of the basins. However, if the actual specific gravity and particle size distributions are not available or the concentration of pollutants in the flow is uncertain, the use of the latter method is not warranted. This is particularly true in the case of combined sewer overflows due to the generally higher percentage of organic solids when compared to stormwater discharges (see Section 2).

The required surface area for oxygen transfer should be based on a surface loading of 5 to 10 lb BOD_5/acre·d (6 to 11 kg/ha·d).

Step 2 and Step 3 are iterative steps. The costs of mass load reduction must be compared with the beneficial uses being protected.

Step 4 - Identify Feasible Pond Sites

Topographic and land use maps of the area may be used to locate potential sites. Land use plans should be reviewed to make sure that conflicts of land use will not occur in the future.

Removal of pollutants from water percolating through a layer of soil is a complex process, and the efficiencies of removal may vary from pollutant to pollutant. Percolation through soil is very effective in removing BOD_5, bacteria, and suspended material. The removal of these pollutants from percolated stormwater is usually complete. Other pollutants, such as some dissolved heavy metals or salts, may be carried into the groundwater or transported via the groundwater to resurface downgradient. The possibilities of groundwater contamination or groundwater transport should be considered when selecting a retention/percolation pond. See the performance section of this chapter for a discussion of treatment mechanisms and efficiencies.

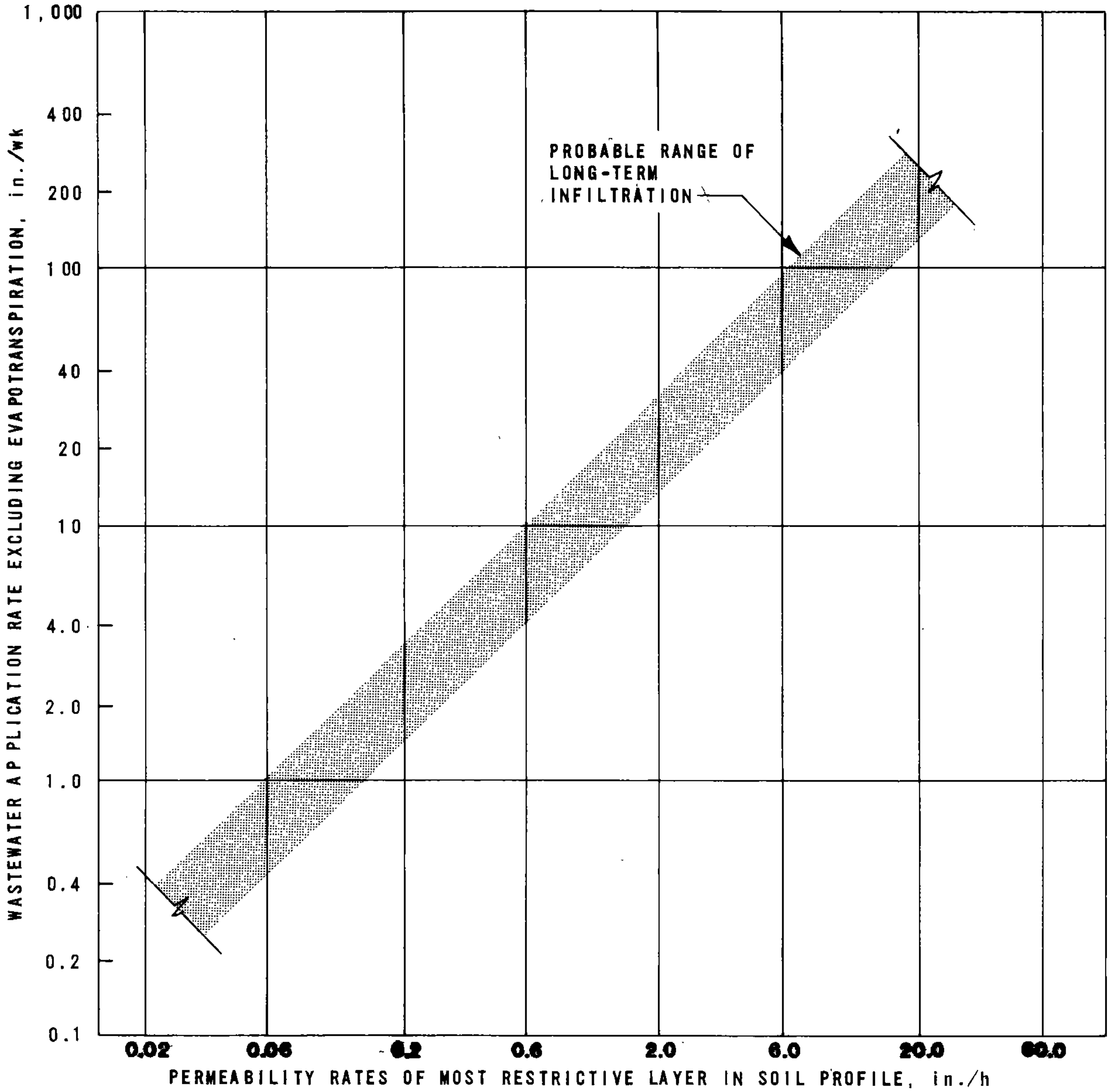
Once the pollutant removal that can be accomplished by sedimentation is determined in Step 3, the additional pollutant removal that can be accomplished by percolation must be determined so that feasible pond sites can be identified. A time step approach using the hydrograph from Step 1 and the associated pollutant loads from Step 2 should be used.

The volume of stormwater that can be percolated is dependent on the volume of water in storage at the end of each time step and the infiltration/percolation rate. Infiltration/percolation also takes place during the storm event whenever there is water in the pond. The volume percolated during each time step is equal to the percolation rate times the length of the time step. But in no case can the percolated volume exceed the stored volume at the beginning of the time step.

The pollutant load removed is equal to the pollutant concentration in the percolated volume multiplied by the percolated volume.

For wet ponds, even though the percolation is continuous the pollutant load varies between dry-and wet-weather values. Also, evaporation should be considered when sizing wet ponds.

Preliminary percolation area sizing of the pond may be performed using the nomograph shown in Figure 19 and soil permeability ranges obtained from Soil Conservation Service soil maps.



PERMEABILITY,* SOIL CONSERVATION SERVICE DESCRIPTIVE TERMS						
VERY SLOW	SLOW	MODERATELY SLOW	MODERATE	MODERATELY RAPID	RAPID	VERY RAPID
< 0.06	0.06–0.20	0.20–0.60	0.60–2.0	2.0–6.0	6.0–20.0	> 20.0

* MEASURED WITH CLEAR WATER

1 in. = 2.54 cm

Figure 19. Soil permeability versus ranges of application rates [3].

The design infiltration rate should be 10 percent of the initial soil permeability value to take into account the decrease in infiltration that will result from surface clogging by suspended solids. The design infiltration rate shown in Figure 19 should be used only for preliminary sizing purposes. Actual field test results should be used for final design of the pond. The required soil/water interface area may be calculated by Equation 5-5.

$$A = \frac{SV}{DI \times ET} \tag{5-5}$$

where A = the soil/water interface area required
SV = the calculated storage volume
DI = the design infiltration rate
ET = the emptying time

The storage volume required is that volume calculated in Step 3. The permeability may be obtained from soil maps of the site and by reading the chart at the bottom of Figure 19.

The emptying time, ET, is the time required to completely percolate the storage volume. Ideally, the entire storage volume should be available at the beginning of each runoff event. However, storm events are random, and the interstorm time will vary. In addition, alternative uses will require that the pond be dry some percentage of time. For purposes of performing a preliminary estimate of bottom area, ET may be estimated for dry ponds by Equation 5-6.

$$ET = (1 - N/100) \times \text{Avg antecedent dry period} \tag{5-6}$$

where Avg antecedent dry period = the average time between the end of one storm and the beginning of the next storm where the runoff volume is equal to or greater than storage volume.

N = the percentage of time during the year that alternative uses require that the pond be dry, %.

For wet ponds, ET is equivalent to average interstorm time. Equation 5-6 may be adjusted to account for seasonal rainfall patterns by calculating average interstorm time and N for the most critical season. For dry ponds, ET should not exceed 7 days for stormwater runoff; for combined sewer overflows, ET should not exceed 3 days.

As many feasible sites as possible should be identified. Sites should then be ranked, based on apparent acceptability. Some subjective judgment must be used since many difficult to quantify factors must be considered, such as land use compatibilities and possible alternative facility uses.

Step 5 - Investigate Most Promising Sites

Beginning with the highest ranked site, soil borings and infiltration and permeability tests of the site should be accomplished. The two preferred methods of infiltration testing are flood basins and ring infiltrometers. Each method is discussed in Appendix C of this manual. Infiltration testing, particularly using ring infiltrometers, should be conducted on the most restrictive soil layer underlying the site within the potential elevation range of the excavated pond bottom.

Again, the measured infiltration rate can be expected to decrease with time as stormwater solids clog the soil surface. The infiltration rate used in design should be 10% of the measured infiltration rate at the soil surface but should not exceed the permeability of the most restrictive soil layer.

Step 6 - Establish Basin Sizes

The next step is to determine a final basin size based on the measured infiltration rate and on a readjusted basin emptying time and percolation area.

The storage volume calculated in Step 3 and the percolation area calculated in Step 4 assumed that, after each runoff event, adequate time for the basin to empty would be available before the next runoff event. In practice, however, events are random and all storage volume may not be available at the beginning of the next storm. Therefore, the basin volume and percolation area must be adjusted to account for this.

The maximum emptying time, ET, and the design infiltration rate, DI, (10% of the measured rate) determine the maximum allowable depth of ponding, d:

$$d = ET \times DI \tag{5-7}$$

For any given storage volume, a direct relationship between available storage volume, ASV, and interstorm period, t, exists:

$$ASV = (A \times DI \times t) \tag{5-8}$$

where A = area of pond bottom

ASV may not exceed the storage volume for dry ponds.

From Step 1, the probability distribution of interstorm periods is known. The interstorm period is related directly to available storage volume by Equation 5-8. Therefore, the probability of occurrence for various available storage volumes is known. Multiplying the probability that the storm will be exceeded by the probability of having the corresponding storage volume available will yield a probability of overflow for each ASV.

By assuming an acceptable level of containment, say 80%, one may obtain the runoff volume controlled by the basin size considered. By performing this calculation for several basin sizes, one may obtain a basin size runoff volume curve for the assumed level of containment.

The design basin size can be selected, based on the percentage of runoff to be contained and the associated waste load reduction (Step 2).

The basin sizing process involves a number of repetitive calculations and can be carried out on a computer.

Note that this process assumes that the storage basin is always full at the end of the preceding storm event. This is a conservative assumption.

The need for a large pond bottom area for infiltration and percolation, due to the existence of a low permeability subsurface soil layer, can sometimes be reduced by installation of underdrains to collect and discharge percolated stormwater.

Step 7 - Design Solids Removal Technique and Facilities

The handling and removal of captured solids within the retention/percolation ponds present the biggest problems in facility operation. In addition, during overflow conditions and sedimentation, short-circuiting and resuspension of previously settled solids may occur, reducing overall basin removal efficiency. Very important aspects of the dry pond design are inlet and outlet structures and solids removal methods and/or devices.

Inlet structures should be designed to provide even distribution of flow across the head of the basin. Devices include overflow weirs, multiple pipe inlets, and hydraulic energy dissipation devices such as stilling walls. Prescreening of stormwater flows and combined sewer overflows is often necessary to reduce the cleanup required if large quantities of paper and other gross solids or floatable materials are present. Compartmentalization of the pond can localize the cleanup of floatables.

As the stormwater flow enters the pond from a channel or conduit, the velocity quickly drops. Since the ability of flowing water to transport heavy solids is directly related to the velocity, a large quantity of the suspended material settles out of the stormwater in the first few feet of the basin. If not removed, a bank of solids may develop, emitting odors as biodegradable materials are anaerobically decomposed. Many flood control basins are constructed with forebays to confine the solids for easier removal. This capture and removal of solids can help extend the period between scarifying of the dry pond bottom or dredging of the wet pond and reduce operational costs.

The forebay may be designed to act as a detention storage/sedimentation basin during small runoff events, with return of the captured flow and solids to the sewers for treatment. A possible inlet/forebay structure is illustrated in Figure 20. Alternatively, a detention storage/sedimentation basin may be used as a pretreatment facility in front of a dry pond.

The width of the forebay should be based on expected changes in flow velocity and settleability of the stormwater, similar to the design approach for storage/sedimentation basins (Section 7).

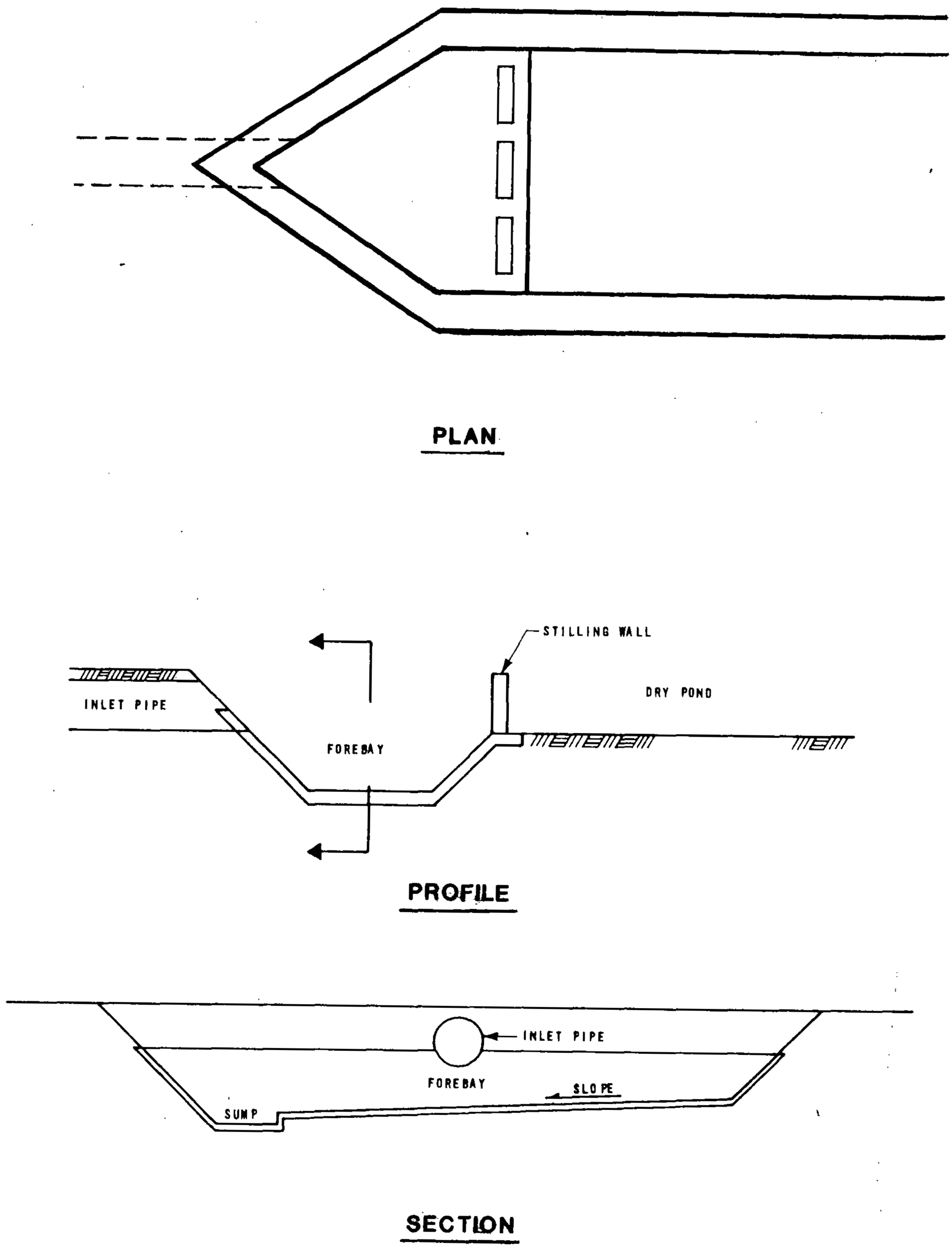


Figure 20. Inlet structure/forebay.

An alternative to the forebay or pretreatment detention basin is to construct the pond in a triangular shape, with inflow at the apex and the overflow along the base. In this way, the drop in velocity is gradual along the length of the basin and the deposited solids are more evenly spread over the basin bottom. The use of baffles or compartments within the pond can improve hydraulic distribution and minimize temperature gradients and salinity stratifications (particularly for combined sewer overflows).

The velocity of the basin flow together with the settling velocity of the suspended particles play predominant roles in the sediment trapping performance of a pond during overflow conditions. The velocity of basin flow depends upon the basin outflow rate. Outflow rates are usually determined by the outflow structure configuration. Commonly used pond outlet forms include overflow weirs, sluice gates, orifices, and spillways. Hydraulic texts should be consulted for the equations to be used in calculating flowrate of the structure selected. The selection of the overflow and its design are usually based on flood discharge requirements.

Step 8 - Determine Pond Configuration

Economical earth construction methods usually dictate that square or rectangular configurations be used with the length not greater than three times the width [8]. Side slopes should be shallow enough (1:3 or less) to allow mowing or other maintenance of any vegetative cover. Many times, however, the final configuration of the retention facility is determined not only by the necessary storage volume and infiltration/ percolation area but also by any alternative uses (recreation, aesthetic, etc.) or physical constraints of the selected site. The method selected for sediment removal or pond bottom maintenance can effect the configuration also.

The overall objective is to provide the best effective retention and pollutant removal facility consistent with the constraints imposed by the site configuration and topography in addition to any other desired uses.

PERFORMANCE

The efficiency of retention ponds in reducing stormwater pollutant loadings depends heavily on the underlying soil as a treatment medium. Soils have been found to be very effective in removing a broad range of pollutants from wastewater (including suspended material, phosphorus, some metals, bacteria and viruses) and the soil also provides a medium for stabilization of oxygen-demanding materials. The effect of wet pond retention on suspended solids for an impoundment at Woodlands, Texas, is shown in Figure 21. The mechanisms of removal include settling, filtering (straining), biological activity, coagulation, adsorption, and chemical reaction. Percolation of wastewater may result in degradation of the groundwater; therefore, it is important to have an understanding of the removal processes at work in soils and removal efficiencies that might be anticipated.

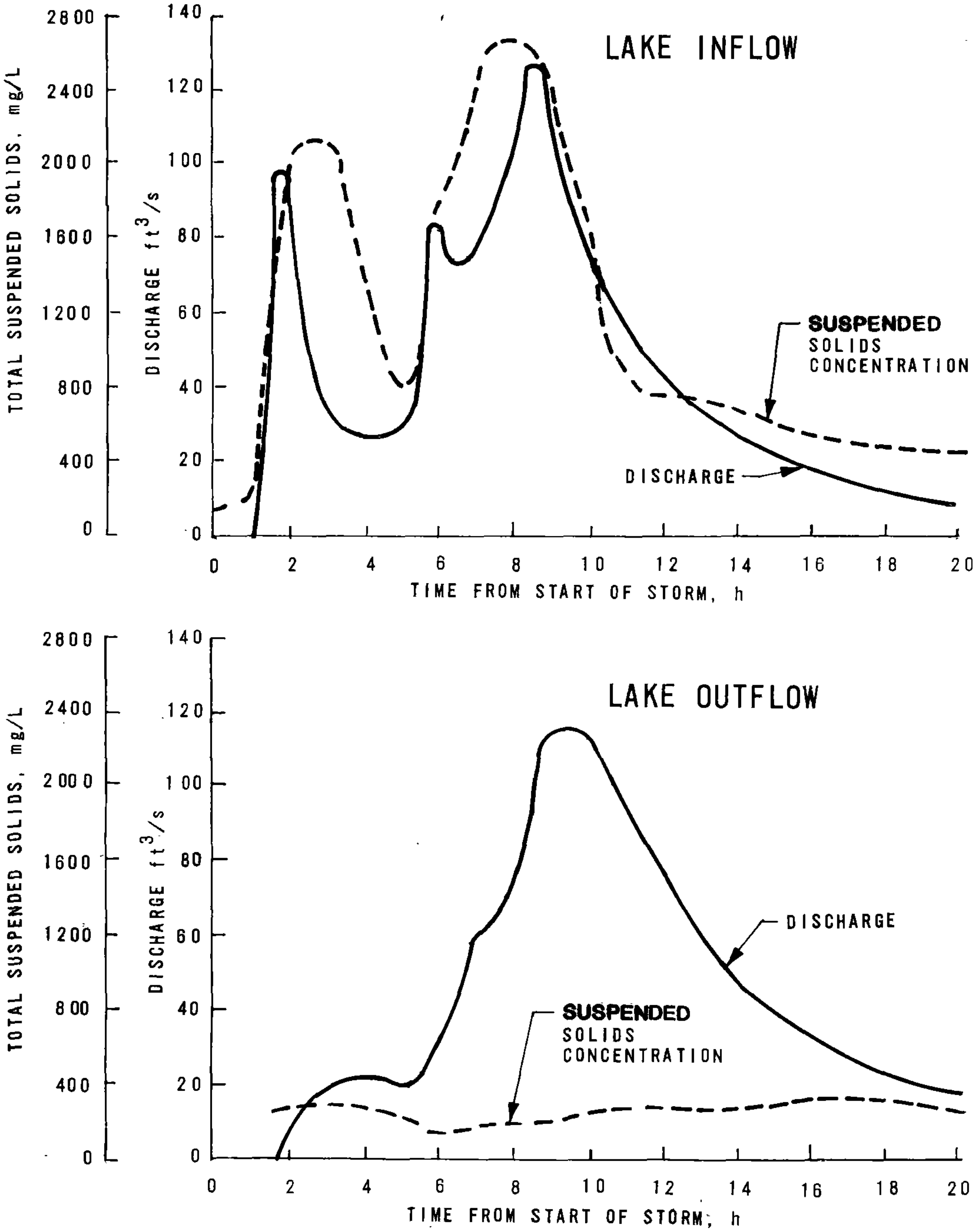


Figure 21. Effect of impoundment on storm runoff in The Woodlands, Texas [9].

Very few investigations of removal efficiencies and treatment performance of soil in percolating stormwater have been conducted. The following discussion is based on observations of land treatment systems for rapid infiltration of municipal wastewater treatment effluent.

Filtration in the soil profile effectively eliminates suspended solids from percolating wastewater. This filtration occurs almost exclusively on the surface. Removed particles tend to fill soil void spaces, further improving the removal by straining. Once retained in the soil profile, biodegradable solids undergo decomposition. Nondegradable and slowly degradable solids, however, tend to gradually build up within the soil, causing clogging and decreasing the infiltration rate. Aerobic decomposition of retained degradable solids and clearing of surface soil pores is enhanced by scarifying or plowing the soil surface.

The ultimate fate of an organic compound in the soil environment depends largely on its ability to be metabolized by soil microorganisms. Microorganisms growing on the soil particles quickly contact and stabilize degradable organic compounds as the wastewater trickles through the soil. If the soil is unsaturated, oxygen will circulate through the soil pores and the stabilization will be aerobic. If the soil is saturated for some period of time, available oxygen may be used up and the process may become anaerobic.

It is important to note than many organic compounds are not susceptible to microbial degradation. Additionally, a variety of organic compounds may be unavailable for microbial or enzymatic decomposition because of environmental factors such as pH, organic matter, moisture content, temperature, aeration, and cation exchange capacity. Other removal mechanisms, in addition to biological stabilization at work on organic molecules in soils, include volatilization, sorption, and chemical degradation. Unless removed by these mechanisms, wastewater organics move through the soil by mass-transport and dispersion and into the groundwater. Concentrations of trace organics in groundwater downstream from spreading basins at Whittier Narrows, California, that receive stormwater, reclaimed wastewater, and surface water are presented in Table 10.

Bacteria, viruses, and parasites present in stormwater discharges and combined sewer overflows may pose a threat to human health due to waterborne disease transmission. Percolation of wastewater through soil can effectively eliminate pathogenic microorganisms. Filtering at the soil surface and at intergrain contacts, and sedimentation and sorption by soil particles are the major removal mechanisms for bacteria. Bouwer and Chaney [10] stated that fecal coliform bacteria are generally removed after 2 to 3 in. (5 to 7.5 cm) of travel in soils. Coarse soils and high rates of application may make 100 ft (30 m) or more of travel necessary for complete removal [11].

Table 10. TRACE ORGANICS IN GROUNDWATER DOWNSTREAM OF SPREADING BASINS OF WHITTIER NARROWS, CALIFORNIA [12]
Concentrations in μg/L

	Wells						
	Unchlorinated					Chlorinated	
Target compound	6-V-W	8-V-W	11-V-W	15-V-W	16-V-W	10-V-W	12-V-W
Vinyl Chloride	<1	<1	<1	<1	<1	<1	<1
Methylene Chloride	2.2	39.2	1.6	3.8	1.6	1.6	1.8
1,1-dichloroethane	<0.1	<0.1	<0.1	<0.1	<0.1	0.8	0.8
Chloroform	2.6	7.2	1.8	1.0	0.4	7.2	<0.4
1,2-dichloroethane	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2
Carbon tetrachloride	--	0.2	0.6	<0.1	<0.2	<0.2	<0.1
Bromodichloromethane	0.2	<0.2	0.2	0.2	3.6	<0.1	0.2
Trichloroethylene	2.3	1.0	1.6	1.2	<0.1	<0.2	<0.2
Dibromochloromethane	<0.1	<0.1	0.4	0.6	<0.1	>50	2.7
1,1,2-trichloroethane	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2
Benzene	0.9	<0.1	<0.2	<0.2	<0.1	<0.2	<0.2
Bromoform	0.4	<0.1	0.4	<0.1	1.2	>40	<0.1
Toluene	0.2	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1
Chlorobenzene	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1
1,4-dichlorobenzene	0.7	<0.2	<0.2	<0.2	0.2	<0.2	0.7
1,2-dichlorobenzene	<0.1	<0.1	<0.1	<0.1	1.2	<0.1	0.9
Tetrachloroethylene	0.8	1.2	1.1	0.4	0.5	0.4	0.4

Unlike bacteria, adsorption is probably the predominant factor in virus removal by soil. Factors such as pH of the media, presence of cations, and presence of ionizable groups on the virus affect this mechanism. Once retained in the soil, viruses survive for up to 6 months [10]. Land treatment sites at which enteric viruses have been detected in the groundwater are listed in Table 11. It should be noted that the systems at Vineland, New Jersey, and Ft. Devens, Massachusetts, operate with undisinfected primary municipal effluent. Factors that may influence bacterial and viral survival in soils are shown in Table 12.

Table 11. REPORTED ISOLATIONS OF VIRUS BENEATH LAND TREATMENT SITES [13]

Site location	Distance of virus migration, ft: Vertical	Distance of virus migration, ft: Horizontal
St. Petersburg, Florida	20	--
Cypress Dome, Florida	10	23
Fort Devens, Massachusetts	60	600
Vineland, New Jersey	55	820
East Meadows, New York	37	10
Holbrook, New York	20	150

Table 12. FACTORS THAT AFFECT THE SURVIVAL OF ENTERIC BACTERIA AND VIRUSES IN SOIL [13]

Factor		Remarks
pH	Bacteria	Shorter survival in acid soils (pH 3 to 5) than in neutral and alkaline soils
	Viruses	Insufficient data
Antagonism from soil microflora	Bacteria	Increased survival time in sterile soil
	Viruses	Insufficient data
Moisture content	Bacteria and viruses	Longer survival in moist soils and during periods of high rainfall
Temperature	Bacteria and viruses	Longer survival at low (winter) temperatures
Sunlight	Bacteria and viruses	Shorter survival at the soil surface
Organic	Bacteria and viruses	Longer survival (regrowth of some types of bacteria when sufficient amounts of organic matter are present)

Organic nitrogen compounds in wastewaters applied to soil are quickly oxidized to the nitrate form under aerobic conditions. Under anaerobic conditions, nitrates may be denitrified and nitrogen removed as a gas.

Denitrification does require anaerobic conditions and the presence of an available carbon source. Land application systems may be operated to maximize denitrification; however, the unpredictability of stormwater makes such operation difficult if not impossible. Therefore, nitrogen removal by percolating stormwater is likely to be poor.

In contrast to nitrogen, the behavior of stormwater applied phosphorus is controlled primarily by chemical, rather than biological reactions. Soluble orthophosphate can be chemically adsorbed onto soil surfaces or directly precipitated. In the adsorption process, orthophosphates react with iron, aluminum, or calcium ions exposed on soil surfaces. Over time, reactions occur that use adsorbed orthophosphate to form phosphate minerals with low solubilities. This, coupled with the creating of new sorption sites, by alternating drying and wetting, regenerates new sites for adsorption. Phosphorus removals can range from 70 to 99%.

In addition to toxic organic pollutant removals discussed above, soils have been effective in reducing the concentrations of trace elements in percolating water over limited periods of time. However, the long-term ability to remove metals is questioned, as soil sorption sites are thought to become saturated. Removal mechanisms for several trace elements are shown in Table 13.

OPERATIONS

As with most stormwater pollutant control facilities, the major operational problems with ponds center around handling of captured solids. Inlet and outlet structures also can present operational problems. Other operational concerns for dry ponds include maintenance of vegetative cover through alternating wetting and drying periods, control of insects, and maximizing availability of the pond for alternative uses.

The problem of solids deposition in intermittently used flood control basins has been recognized for some time. Often, such facilities are constructed with forebays in an attempt to concentrate the solids for easier removal. Dry ponds may be constructed in series with a detention storage/sedimentation facility ahead of the pond. During small volume runoff events, the heaviest solids are captured in the detention facility and returned to the sewers for treatment. The settled overflow is allowed to infiltrate and percolate from the ponds. For ponds in which stormwater control is the exclusive use, frequent discing of the pond bottom will aid aerobic decomposition of biodegradable solids and disperse the captured solids through a thicker layer of the soil. This will reduce surface clogging. No matter what the alternative use of the dry ponds, discing or scarifying of the bottom surface must be practiced periodically to maintain infiltration capacity.

Table 13. REMOVAL MECHANISMS OF TRACE ELEMENTS IN SOIL [13]

Principal forms in soil		
Trace elements	Solution	Principal removal mechanism
Ag (silver)	Ag+	Precipitation
As (arsenic)	AsO_4^{-3}	Strong association with clay fraction of soil
Ba (barium)	Ba^{+2}	Precipitation, sorption into metal oxides and hydroxides
Cd (cadmium)	Cd^{+2}, complexes, chelates	Ion exchange, sorption, precipitation
Co (cobalt)	Co^{+2}, Co^{+3}	Surface sorption, surface complex ion formation, lattice penetration, ion exchange, chelation, precipitation
Cr (chromium)	Cr^{+3}, Cr^{+6}, $Cr_2O_9^{-2}$, CrO_4^{-2}	Sorption, precipitation, ion exchange
Cu (copper)	Cu^{+2}, $Cu(OH)^+$, anionic chelates	Surface sorption, surface complex ion formation, ion exchange, chelation
F (fluorine)	F^-	Sorption, precipitation
Fe (iron)	Fe^{+2}, Fe^{+3}, polymeric forms	surface sorption, surface complex ion
Hg (mercury)	Hg^o, HgS, $HgCl_3^-$, $HgCl_4^{-2}$, CH_3Hg^+, Hg^{+2}	Volatilization, sorption, chemical and microbial degradation
Mn (manganese)	Mn^{+2}	Surface sorption, surface complex ion formation, ion exchange, chelation, precipitation
Ni (nickel)	Ni^{+2}	Surface sorption, ion exchange, chelation
Pb (lead)	Pb^{+2}	Surface sorption, ion exchange, chelation, precipitation
Se (selenium)	SeO_3^{-2}, SeO_4^{-2}	Ferric oxide-ferric selenite complexation
Zn (zinc)	Zn^{+2}, complexes, chelates	Surface sorption, surface complex ion formation, lattice penetration, ion exchange, chelation, precipitation

Vegetative cover on dry ponds serves a variety of purposes. The most obvious is to make the pond more aesthetically acceptable and to allow alternative uses such as athletic fields. In addition, vegetation can supply some removal of pollutants, particularly nitrogen, by plant uptake and vegetation helps to maintain high infiltration rates. The effects of vegetation on infiltration rates are illustrated in Figure 22. The vegetation prevents reduction of the infiltration capacity through compaction of the soil surface. Successfully used vegetation includes fescue, perennial rye, and bermudagrass.

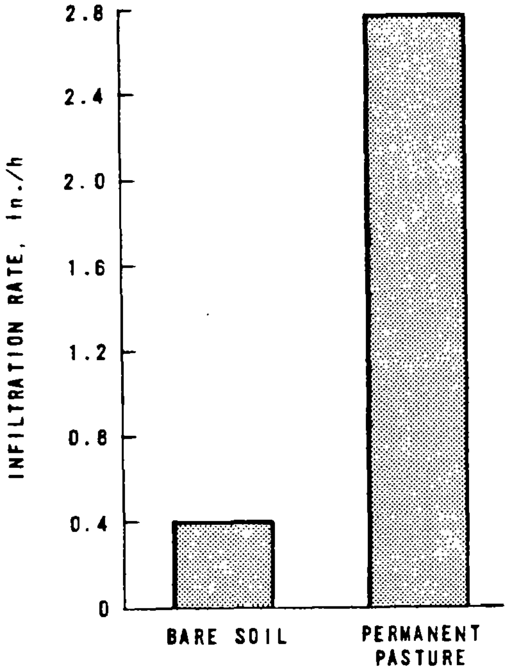


Figure 22. Effect of vegetation on soil infiltration rates [3].

The alternative uses and the surrounding land uses will, to a large extent, determine the operational schedule and requirements. Heavy deposits of organic solids may produce odors as they decompose and should be prevented. The obvious presence of deposited solids may also be visually objectionable.

The major operations considerations for wet ponds are sediment handling and removal, control of floating materials, erosion of the pond banks, algae and aquatic weed control, prevention of nuisance odors, and control of insects.

As was the case with dry ponds, the major operational problem with wet ponds is sediment handling and removal. Deposition of materials near the pond inlets can result in buildup of sludge banks, generation of odors as the deposited solids decompose, and loss of pond volume. In addition, the infiltration rate of water through the pond bottom may decrease.

Because wet ponds do contain water all the time, removal of sludge banks and scarifying of the pond bottom is much more difficult than for dry ponds. Construction of a forebay or sedimentation basin in front of the wet pond is the most effective method of solids control. The pond may also be periodically dredged or even drained and the solids removed.

Floating materials in the stormwater may also present an operational problem. Floating materials may clog pond outlets or overflows, may interfere with oxygen transfer at the pond surface (resulting in anaerobic conditions and odors), and may provide conditions suitable to insect breeding. Floating materials are also unsightly. Floating materials may be controlled by prescreening and/or installation of a floating boom (see Figure 23) near the pond inlet. The boom must be cleaned after each runoff event.

Figure 23. Floating material trapped by log boom.

Erosion of the pond banks may result from wave action particularly if the pond is large and exposed to winds. Erosion problems are primarily the result of neglect of surveillance and maintenance. The maintenance of proper grass cover and riprap minimizes the problem.

Aquatic plants have both desirable and undesirable effects in ponds. The weeds may exert a significant oxygen demand as they decompose. In addition, they are often unsightly. They provide a suitable habitat for insect breeding. Woody plants and trees tend to weaken dikes by their root growth. On the other hand, marsh treatment systems rely on aquatic plants to uptake nutrients from wastewater and to promote settling by enhancing quiescent conditions. Maintenance of 2 ft (0.6 m) of water depth will discourage growth of weeds. Weeds growing from the pond bottom should be pulled and removed, rather than sprayed or cut, since the decaying material may exert a significant oxygen demand.

Control of algae growth is another important operational consideration. Algae blooms, sudden and extreme growth usually of blue-green algae, tend to die-off with a rapidity equal to that of the growth. The dead algae then furnish an extremely large and sudden oxygen demand, frequently producing anaerobic conditions and odor problems. Algae blooms may be controlled by the use of chemicals or certain algaecides approved by the USEPA.

Insects commonly found near retention ponds include mosquitoes, midges, beetles, and dragonflies. Insect generation occurs in sheltered areas or quiescent portions of the pond where there may be substantial growth of rooted plants or layers of scum. The basic measures for insect control are control of weeds and scum and prevention of stagnant conditions. Insecticides also may be used, as shown in Table 14. If insecticides are used, 1 or 2 days of contact time is usually required. Prolonged contact periods will lessen the dose required for equal effect.

Table 14. SOME INSECTICIDES USED FOR LAGOON INSECT CONTROL [7]

Insect	Insecticide	Application rate
Culex Mosquitoes	Dursban	1 mg/L
	Naled	1 mg/L
	Fenthion	1 mg/L
	Abate	1 mg/L
	Diesel oil	6 to 8 gal/acre
	Malathion	2% sprayed around edge
	Abate	2% sprayed around edge
	BHC	Dust, 3% gamma isomer
Midges	Fenthion (Baytex)	As directed on package
	Abate	As directed on package
	Sursban	As directed on package
"Shrimp-like" insects; algal predators	Dibrom-8	As directed on package

COSTS

The costs of constructing dry or wet retention/percolation ponds may be estimated using the curves shown in Figures 24 and 25. The cost curves are based on construction costs for rapid infiltration basins and storage ponds for disposal of domestic wastewater. Land costs are not included. These costs should be used for preliminary estimates only. Actual costs depend on the initial conditions of the site and the dual uses of the pond, particularly recreation uses. Operation and maintenance costs for either type of basin are highly site specific and depend on stormwater pollutant concentrations, frequency of runoff events, and dual facility uses.

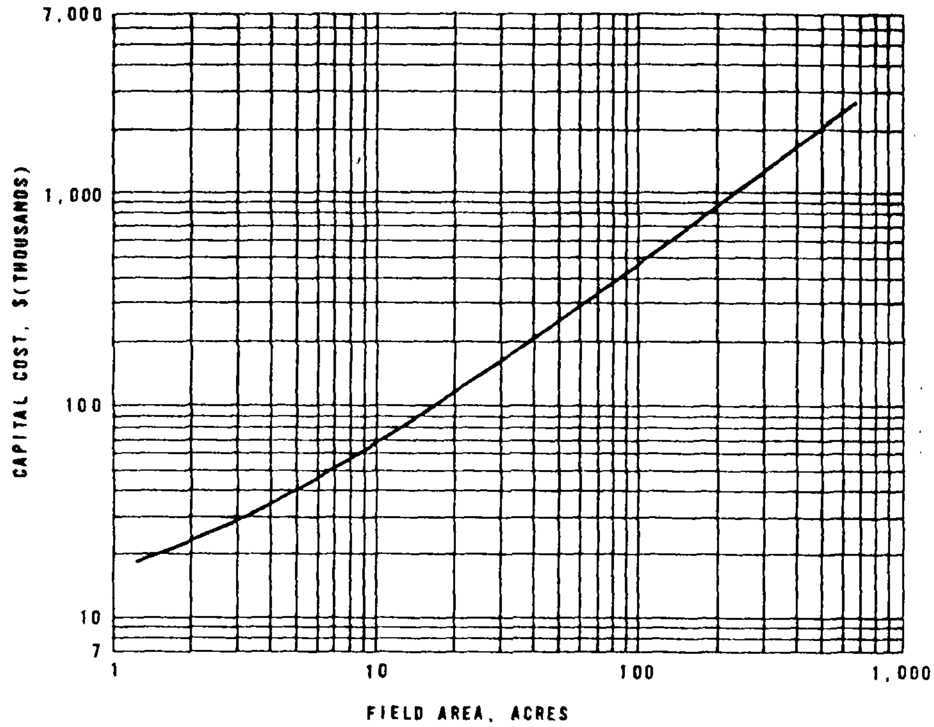


Figure 24. Cost of dry pond construction, ENR 4000 [14].

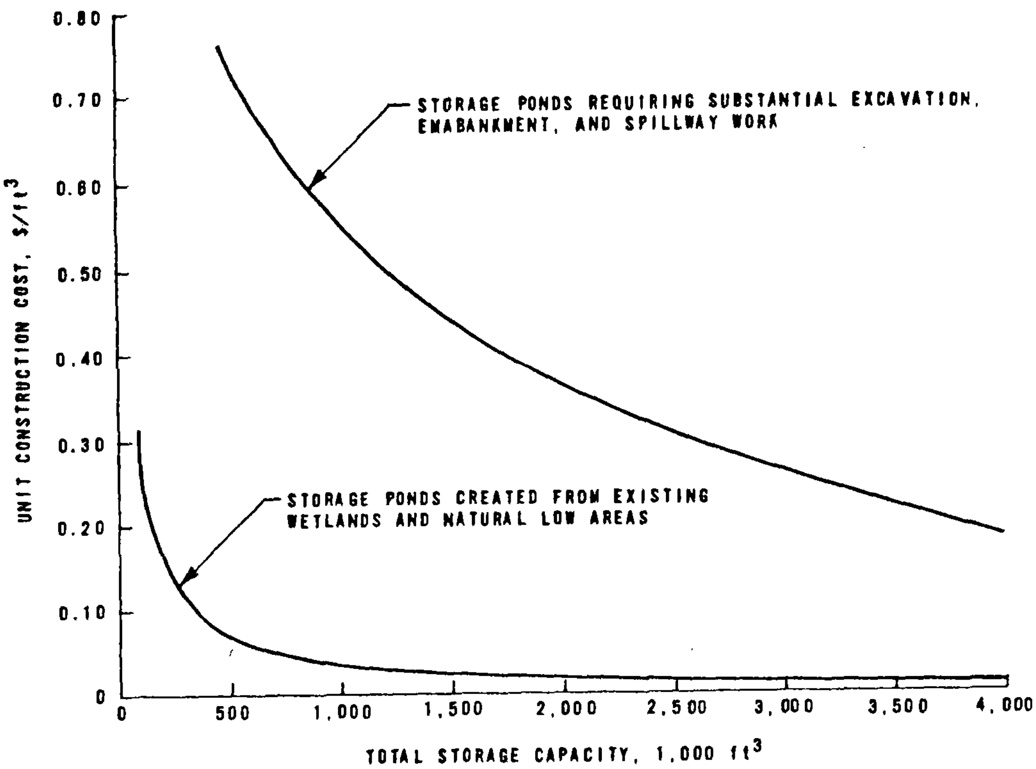


Figure 25. Storage pond construction costs, ENR 4000 [9].

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Section 6

DESIGN OF DETENTION FACILITIES

INTRODUCTION

Many communities throughout North America have stormwater runoff and flooding problems according to a recent APWA survey [1]. For example, nearly half of the respondents stated that disposal of stormwater runoff is a problem, with basement flooding a serious problem experienced in more than half the communities. Some of the problems identified include soil erosion; sedimentation; flooding of commercial and industrial property, places of human habitation, streets, intersections, and highway underpasses; bridge and street washouts; recurring basement backups from surcharged sanitary sewers, attributable to illicit roof and foundation drain connections, and from combined sewers; inflow and infiltration of stormwater into sanitary sewers; wastewater treatment plant bypassing and overflows of stormwater and wastes from combined sewers. The consequences of these problems include loss of human life and damage to real and personal property; health hazards; delays of emergency vehicles and workers reaching places of employment; cleanup demands on municipalities and citizens; adverse effects on the aesthetics of natural areas and urban environments; personal inconvenience; pollution threats to groundwater supplies; disruption of ecological balances; disturbance of wildlife habitats; loss of animal life; and economic losses associated with the problems identified above [2].

To alleviate these problems, planning for stormwater detention is often part of the overall stormwater management plan. For example, of the drainage master plans reported, more than half included the development of detention facilities. Two hundred nineteen public agencies reported having detention facilities; the number and type are listed in Table 15. A total of 12,683 facilities were reported, an average of 58 per community. Nearly 40% of those communities without detention facilities said that facilities are being built, are in the planning stage, or have been considered and are a priority item for the near future.

Objectives reported by the public agencies for establishing detention facilities are given in Table 16 [1]. Reducing the cost of drainage systems and reducing pollution from stormwater are two of the top seven objectives on the list.

Stormwater detention storage delays excess runoff and attenuates peak flows in the surface drainage system. This storage, because of sedimentation during detention, can also be considered a treatment process.

Table 15. DETENTION FACILITIES IN USE IN THE UNITED STATES AND CANADA [1]

Type of facility	Total in use No.	Total in use %	Private No.	Private %	Public No.	Public %
Dry basin	6,053	47.8	4,913	81	1,140	19
Parking lot	3,134	24.7	2,982	95	152	5
Pond	2,382	18.8	1,199	50	1,183	50
Rooftop storage	694	5.5	644	93	50	7
Underground tank	160	1.3	142	89	18	11
Oversized sewer	135	1.0	83	61	52	39
Underground tunnel	9	0.1	8	89	1	11
Other	116	0.9	64	55	52	45
Total	12,683		10,035	79	2,648	21

Table 16. OBJECTIVES IN REQUIRING DETENTION [1]

Objective	Rank[a]
Reduce downstream flooding	100
Reduce cost of drainage systems	71
Reduce onsite flooding	70
Reduce soil erosion	66
Capture silt	64
Improve onsite drainage	63
Reduce pollution from stormwater	56
Improve aesthetics	53
Enhance recreational opportunities	51
Replenish groundwater	42
Supplement domestic water supply	36
Capture water for irrigation	35
Other	22

a. In order of importance using 100 as "most important."

The sizing of detention facilities also requires consideration of additional parameters such as design storms, site constraints, and outflow rates. Whenever possible, the "design storm" should consist of a continuous historical or synthesized rainfall record that is typical of any long-term rainfall record. The use of statistical rainfall intensity-duration-frequency relationships should be avoided except for an initial rough-cut estimate since this approach does not account for the effects of short intervals between storms. Actual historical rainfall records selected may be based on the hourly intensity, storm duration, total rainfall for the storm, or any combination of these. Site constraints to be considered for detention storage facilities include tributary area, topography, local land use, and area available for the structure or basin. The outflow rate from the detention facility may be based on the capacity (size and slope) of the drainage channel or conduit downstream, the capacity of a treatment facility downstream, or a regulatory limitation. An example of a regulatory limitation might be that the rate of runoff from a developed piece of property be no more than that before the property was developed.

Onsite Detention

The concept of onsite detention was presented in Section 3. Typical examples of onsite storage include rooftops, plazas, parking lots and streets, drainage swales, blue-green storage, check dams, underground structures, and multipurpose detention reservoirs.

Many municipalities, having faced the results of increased stormwater runoff volumes and rates from urban development, are now enacting ordinances requiring developers to limit the rate of stormwater runoff from developed areas. An example of such an ordinance is that enacted by the Metropolitan Sanitary District of Greater Chicago (MSDGC).

The MSDGC ordinance limits the peak rate of runoff from newly developed areas to that which would occur on the land in its undeveloped state as a result of the 3-year frequency rain storm. Any amount exceeding that must be stored for gradual release. Detention facilities must be designed to handle the 100-year storm without flooding.

In-System Detention Storage

The concept of in-system detention storage was presented in Section 3. Typical examples of in-system detention include inline storage (the use of available volume in trunk sewers, interceptors, and tunnels to store stormwater or combined sewage) and offline storage (open or covered basins, caverns, mined labryinths, and lined or unlined tunnels).

Overflows from combined sewer systems without stormwater detention controls generally occur whenever the rainfall intensity after the time of concentration for the tributary area exceeds 0.02 to 0.03 in./h (0.05 to 0.07 cm/h). This occurs because the peak treatment rate at plants serving combined sewer systems is usually about 1.5 times the dry-weather flow. Because combined sewers are designed to carry maximum flows occurring, say, once in 5 years (50 to 100 times the average dry-weather flow), during most storms there will be considerable unused volume within the major conduits.

Inline storage is provided by restricting the flow with static or dynamic regulators. The installation of regulator devices can create significant inline storage. Static regulators, such as the Hydrobrake, Steinscruv, bulkheads with orifices, weirs, etc., can be used to generate inline storage without controls.

Dynamic regulators, such as sluice gates, Fabridams, etc., usually require sophisticated monitoring and control systems. Dynamic regulators that operate based on interceptor capacity can reduce overflows without sophisticated monitoring and control systems.

Large inline storage installations such as Seattle's CATAD system or Minneapolis-St. Paul's use computers to control multiple dynamic regulators [3, 4]. These systems include (1) remote flow and level sensors; (2) signal transmission; (3) display and data logging; (4) centralized control capability; and (5) in the case of fully automated control, a computer program capable of making decisions and executing control options.

Offline storage as applied in Chicago [5], New York City [6], and Milwaukee [7], for example, does not require such sophisticated controls. These systems generally use pumping stations for controlling the discharge. Thus, sophisticated computer control of the offline storage is not required.

Hybrid storage, such as that used in San Francisco [8], incorporates inline storage tunnels (in effect, greatly oversized transport conduits) with a pump station at the end for discharge control. The utilization of the storage is controlled by the water level settings for the pump controls at the pump station.

DESIGN CONSIDERATIONS

Urban stormwater runoff and combined sewer overflows can be controlled through implementation of storage as a means of source control (onsite detention) or in-system control. Functionally, the application of onsite detention differs little from in-system storage other than the location where the storage occurs. However, while onsite detention is used primarily to minimize the cost of constructing new storm sewers to serve a developing area, in-system storage is generally used to decrease the frequency and volume of overflows from combined sewer systems. Offline storage can be used to selectively capture and direct to the treatment plant a portion of the stormflow (i.e., a first flush contaminant load).

Factors to be considered in the design of onsite storage facilities are (1) tributary area, (2) storage area and volume, (3) structural integrity, and (4) responsibility of the owner. Factors to be considered in the design of in-system storage facilities are (1) size and slope of sewers, (2) peak flowrates, (3) controls, and (4) resuspension of sediment.

Tributary Area

The size of the tributary area determines the volume of water from a given storm that will have to pass the discharge point. The runoff volume is a function of the amount of rainfall, the extent and nature of the topography and land cover, and the size of the tributary area.

In the cases of rooftop and parking lot storage, the tributary area is usually the actual rooftop or parking lot surface area (i.e., no additional tributary area). However, the tributary area for plaza, underground structure, and multipurpose detention facilities usually includes additional area surrounding the facility itself.

Storage Area and Volume

The area and volume available for detention storage depend on the topography of the particular site. In the case of rooftops, the area available is fixed by the geometry of the building. The volume is limited by the rooftop area and the depth of water that can be supported without endangering the structure.

For parking lots, plazas, and multipurpose detention reservoirs, the storage area is less well defined since all or only portions of the area may be used depending on the nature of the site or the desires of the builder. There is usually more latitude available for increasing the detention volume for these facilities by adjusting depth to which the water is allowed to pond.

Underground storage structures may include concrete, fiber glass, or metal tanks and pipe bundles for storage. The storage volume depends on the surface area and depth of the tank or on the diameter and length of pipe bundle. The desired storage volume can be accommodated by varying the dimensions as necessary. In most cases, the depth of such structures is limited by the topography and the location of the sewer to which the structure must discharge.

Structural Considerations

Each of these facilities requires somewhat different structural considerations. Rooftops are limited by the building code design load and the need to prevent leaks into the structure below. The pavement and base for parking lots must be carefully designed and constructed. The structural integrity of the pavement can be jeopardized and the service life drastically reduced because of the presence of water in the pavement base. Plazas, underground structures, and multipurpose detention reservoirs must be designed to allow access for sediment and debris removal. Outlet structures must also be designed to fit the specific application.

Responsibility of the Owner

Even when the designer has provided for maintenance of a detention facility, it is not always easy to determine who is responsible for performing the maintenance. For rooftop detention or for a basin or tank serving an apartment complex or a commercial complex, the ownership and the maintenance responsibility is easily determined. However, for the other types of facilities, this may not be true. A basin may be owned by a number of individuals as part of their house lots. The homeowners, individually, are not a legal entity that can be forced to maintain such a structure. To prevent this, a developer can donate the detention facility to the municipality, which must then provide the maintenance.

Slope and Size of Sewers

To make the most effective use of the unused volume in combined sewers, the conduits used for inline storage should be as large as possible and have relatively flat grades. The flatter the grade, the farther upstream the backwater effect created by the flow control device will extend. Also, the larger the diameter of the conduit, the more storage volume is available per unit length.

Frequently, inline storage is implemented at existing overflow or diversion structures because the conduits are large (and usually relatively flat) at such locations. However, inline storage can be implemented anywhere in the system it is feasible and an appropriate control device can be installed.

Peak Flows

Usually, it is necessary to allow overflow structures to pass the design flow unrestricted when required to prevent surcharging of the sewer system and ponding of runoff on streets. Thus, in-system storage may be implemented at all times when the flow is less than the designed capacity of the sewer. For small storms, the full flow capacity of the sewer may never be reached so that in-system storage is affected throughout the storm.

However, when in-system storage is used in conjunction with onsite storage, some trading-off of surcharging and street or parking lot ponding may be worthwhile and cost effective for combined sewer overflow control or stormwater attenuation. In this case, in-system storage can be used throughout the storm.

Controls

Static regulators usually require no controls at all. The regulator is designed to operate in a single manner with no outside manipulation. These regulators are designed and sized to provide control and operate over a fixed range. To change the range or control strategy requires replacement of the regulator itself.

The control system for effecting in-system storage with dynamic regulators is usually quite sophisticated. Not only are controls required to operate the regulator itself but also for monitoring conditions upstream and downstream of the regulator. This requires that level sensors and overflow detectors be used as a minimum. It may also require rain gage networks, water quality sensors, and flowmeters. If more than one in-system storage location is involved, it may also require a computer to monitor and control the operation of the storage to maximize its effectiveness.

Resuspension of Sediment

One side effect of in-system storage is the settling of suspended material in the sewer as the flow velocity is reduced. This is a definite problem in combined sewer systems, and especially where offline storage is used. To prevent reduced flow capacity problems later on, the sediment must be resuspended and transported to the treatment plant or to the discharge location on separate storm drain systems. The operation of inline storage tends to self-remedy this situation. During periods when there is high flow (approaching peak design flowrate) in the sewer during a storm, the velocity is usually sufficiently high to resuspend and transport any sediment. Sediment can be resuspended also by selectively releasing upstream in-system storage that is centrally controlled.

For offline storage, resuspension of sediment can be a definite problem unless special design considerations are included. Special flushing flows may be required following a storm if flow velocities generated during dewatering of the facility are not sufficient to resuspend and transport the sediment. A portion of the pumped discharge can be recycled to the upstream end of the offline storage unit to resuspend the sediment.

DESIGN PROCEDURE/EXAMPLES

A suggested design methodology for onsite (source control) storage was shown in Figure 9 and for in-system storage was shown in Figure 10. The two methodologies are very similar and can be combined together in the discussion presented in this section. Each of the indicated steps is discussed below and examples are introduced where applicable.

Step 1 - Identify Functional Requirements

Onsite. As noted previously, the intended operational function of an onsite storage facility determines its design emphasis. The design emphasis can also be dictated by the type of development either existing on or planned for the site.

Information that must be determined at this point includes:

- The type of development existing on or planned for the site
- The reason that stormwater detention is needed or desired

The type of development greatly affects the types of stormwater detention facilities that can be employed for the site. Commercial complexes can incorporate combinations of rooftop, parking lot, plaza, and underground structures for stormwater detention. Residential developments usually incorporate rooftop detention (only if flat roofs are used), underground structures, and multipurpose reservoirs.

In-System. The frequency with which overflows to the receiving water occur at various locations in the sewer system determines the design emphasis for in-system storage facilities. Locations where frequent overflows occur or where only small amounts of rainfall initiate overflows are prime candidates for in-system storage implementation.

Identification of the need for overflow control is usually based on one or more of the following:

- Overflow frequency reduction
- Overflow volume reduction
- Overflow quality improvement

Although in-system storage is used most frequently on combined sewer systems, it can also be used on separate storm systems. Provision must be made for removal of any sediment since, unlike a combined sewer system, the stored flow does not receive treatment before discharge to the receiving water. Stormwater volume or frequency of discharge to the receiving water can be reduced if the stored stormwater is later used for groundwater recharge or some other land application use.

Step 2 - Identify Site Constraints

Sites for onsite and in-system storage facilities should be identified and cataloged with respect to at least the following criteria:

- Accessibility to the channel, sewer, or interceptor to which it discharges or to the discharge or overflow point.
- Total area usable for storage (dimensions, configuration, topography) including any area needed for construction and operation of any necessary controls.
- Hydraulic and hydrologic data on rainfall intensity, flow levels in the conduits or channels to which flow is discharged, and allowable discharge rate for onsite storage. Hydraulic data on receiving water levels at the overflow point, flow depth ranges and capacities for the trunk sewers and interceptor, any water level stage and pumping requirements for the proposed facility, stage and corresponding storage volume within the sewer system, and the frequency and volume of overflows for in-system storage.
- Environmental setting such as proximity to residences or other structures, local and surrounding land uses, and visual impacts.
- Geotechnical conditions that could affect load bearing capacity, side slope stability, hazards to adjacent structures and utilities, and groundwater supplies.
- Structural requirements.
- Accessibility to utility services and construction and operation activities.

Step 3 - Establish Basis of Design

In the past, several design methodologies have been used for design of storage facilities. However, the validity of these methods must be measured against how well they reflect the hydrologic cycle and whether or not they include an inflow hydrograph, a depth-storage relationship, a depth-outflow relationship, and a routing routine [2].

More than 45 different methods of predicting runoff rates and developing inflow hydrographs were reported in the APWA survey [1]. The rational formula, the most commonly accepted method, was approved for use by 80% of the respondees even though the method yields only a peak flowrate. The Soil Conservation Service curve number method (the fourth most popular by the respondees) has been used even though it considers only the 100-year, 24-hour storm and watersheds smaller than 2,000 acres (810 ha) [9]. Some methods are applicable only to certain types of situations. Some methods cannot be used on watersheds containing several detention facilities because they handle only one detention facility at a time. Only hydrologic methods that include a channel routing routine can be used for watersheds where channel storage has an effect on the shape of the hydrograph.

The general approach to the design of any detention facility (for either onsite control or in-system control) is basically the same as that described previously for a retention facility in Section 5; it is a storage reservoir routing problem. This applies to all forms of detention facilities. The use of a reservoir routing approach can be used for the design or evaluation of any storage facility. All require an inlet hydrograph, a depth-storage relationship, and a depth-discharge relationship. Some forms may also require a channel storage evaluation or other specialized approaches for the analysis.

The purpose of this step is to determine the inflow rate, allowable discharge rate, pollutant loadings, and storage volume needed to evaluate the effectiveness in meeting the requirements established in Step 1.

Onsite. The allowable discharge rate may be based on the capacity of the conduit or channel at the discharge location, or on building regulations. In the latter case, for example, a municipal ordinance may limit the discharge to that occurring before development occurs.

Specific criteria to be considered during the design and siting of various kinds of detention facilities are described here.

Parking Lots and Streets. The parking area should be graded to create multiple storage areas like saucers. At each low point, a catchbasin or inlet is used to control the outflow. The outflow control can be accomplished either by restricting the size of the outlet pipe or by using a special cover with drilled holes. As a rule, the maximum depth of the detained water at the low point should not exceed 12 in. (30 cm). Thus, the depth of the stored water varies between zero at the edges to 12 in. (30 cm) at the low point.

Almost half of the public agencies responding to the APWA survey indicated that they permit designs to provide for some street flooding. Flooding depths ranging from 6 to 8 in. (15 to 20 cm) were most commonly permitted. The full range of responses is shown in Table 17.

Table 17. DEPTH OF FLOODING ALLOWED ON STREETS [1]

Depth of flooding permitted, in.	Responses	
	No.	%
0	138	42
1-2	14	4
3-5	34	10
6-8	74	23
9-17	16	5
18 or more	6	2
No answer	43	13
Total	325	

Roof Storage. By placing a parapet all around the edge of a flat roof, stormwater may be stored on the roof without concern for the structural integrity of the roof. Most building codes require roofs to withstand 20 to 30 lb/ft^2 (958 to 1,436 N/m^2) live loads [10]. This is equivalent to 4 to 6 in. (10 to 15 cm) of standing water. The detention is controlled by a drain ring set around the roof drains. As the roof begins to pond, flow is controlled by orifices or slits in the ring; extreme flows overflow the ring to prevent structural damage to the roof.

Multipurpose Detention Basins. Multipurpose basins are usually used to store stormwater near the site where it is generated. The required basin size is determined by calculations based on the design storm. Such basins are generally 3 to 5 ft (0.8 to 1.5 m) deep. To prevent water standing in the basin between storms, the invert of the outlet structure is at the same elevation as the bottom of the basin. The outlet structure should incorporate not only an orifice for controlling the outflow rate but also an overflow grating to allow discharge if the orifice is blocked with debris. Typically, a 1 to 2 ft (0.3 to 0.6 m) freeboard should be provided. Sides should be sloped (generally 1-1.5:1 or flatter) and planted with grass that can withstand periodic flooding. These basins can be used as baseball or football fields, tennis courts, or playgrounds. In an Oak Lawn, Illinois, detention basin, a concrete paved area (antierosion section at the inlet) is flooded during winter months to serve as an ice-skating rink [11].

Underground Structures. Concrete, fiber glass, or metal storage tanks can be constructed underground to serve as detention facilities. Such structures may be located beneath parking lots, buildings, or sidewalks and planter strips. Oversized storm sewer pipes can be used in place of storage tanks. Access must be provided to allow removal of any debris that collects in the tanks.

Plazas. The basic design approach for plaza storage is the same as for other forms of detention. The outlet must be construced to allow runoff to accumulate during peak storm conditions. The depth that can accumulate on plazas should be limited to about 0.75 in. (1.9 cm) so pedestrians can still pass, but it is possible to design plazas so that portions can be flooded without inconvenience [12].

In-System. Representative inflow characteristics may be developed as for onsite storage facilities or they may be developed from dry-weather flow data and analyses of historical overflow samples. Direct field measurements may be required.

Storage requirements for the drainage areas must be determined. The relative effectiveness of storage related to overflow volume and/or frequency must be evaluated for each area. The areas must be ranked by relative effectiveness to establish the priority for further evaluation.

The alternative methods available for controlling the in-system storage must be reviewed. The storage can be in multiple, discrete units or integrated into a centralized location. For example, a long, large diameter pipe located in the parking strip along a street with several catchbasins discharging to it can be used to provide storage. Discharge from the storage to the sewer can be controlled by an orifice or a Hydrobrake, for example. Many discrete units such as this could be used to provide the total storage volume needed.

A single, centralized facility could be used to provide the needed storage also. A large, lined basin located adjacent to the sewer (as is the case at the Spring Creek facility in New York City [6]) can be used. For each site, a control method must be selected that will satisfy the needs for that particular site. If more than one site utilizing dynamic control will be involved in the in-system storage system, the means for monitoring conditions at each site and for coordinating the controls to optimize the effectiveness must be selected. This could include simple supervisory control by an operator or full computerized control.

Provisions should be incorporated to resuspend and transport any sediment deposited during in-system storage. This can be accomplished by (1) ensuring that the dry-weather flow has sufficient velocity to resuspend the sediment, (2) sequential release of stored stormwater, or (3) intoducing flushing water to the sewer system. Also, auxiliary controls within the sewer to redirect flow to a single barrel of a multibarrel sewer may be required. The quantity of sediment in a sewer at the end of a storm is a function of the quantity and characteristics of suspended solids in the flow, the length of time the in-system storage was in operation, and the hydraulic characteristics of the sewer.

Step 4 - Select Storage Option(s) and Locations

This step requires that one or more storage methods and sites be selected to meet the functional requirements from Step 1, the site constraints identified in Step 2, and the design criteria established in Step 3.

Onsite. Depending on the type of development contemplated or existing, one or more different storage options may be needed to develop the required storage volume. For example, a combination of rooftop and surface detention might be required for a single-family residential site to meet the discharge requirements. In addition, consideration must be given to access to the storage unit for cleaning. Leaves, twigs, grass, dust, and eroded soil are only some of the items that find their way into storage facilities in various quantities. Depending on the storage option selected, provision must be made to remove these sediments and debris.

In-System. Based on the results of the first three steps, the alternative approaches and sites should be ranked. Detailed evaluation of the sites should be performed in accordance with this priority ranking. The operational concept, storage control method, and projected effectiveness should be determined for each specific site, based on the functional requirement for that site. Operational concepts to be considered should include:

- Storm anticipation so that runoff from short duration storms or "spot cell" storms can be completely captured and treated.

- Selective detention of flow from portions of the system to allow sewer inspection, maintenance, or modification.

- Selective overflowing at points that will have the least effect on the receiving waters.

- First flush interception of the more heavily contaminated stormwater from the early part of a storm.

Auxiliary systems for sediment transport and regulator control must be determined. This should include air handling and odor control, energy (power, lighting, heating), and instrumentation for controls. Availability of utilities needed to operate the storage facility must be assessed.

In addition, an operation plan and a maintenance schedule should be prepared at this time.

Step 5 - Estimate Costs and Cost Sensitivities

Onsite. Detailed cost estimates should be prepared to determine the least cost option(s) to be incorporated. These costs can then be compared to the cost difference between providing an outfall sewer able to convey the peak runoff (usually based on a 5 or 10 year design storm) with that to convey the reduced flow.

In some cases, no additional outfall sewers may be required. Then, the objective is to minimize the onsite storage cost by selecting the options that result in the lowest construction cost.

In-System. Once the location or locations have been selected and a preliminary design of the in-system storage unit(s) completed, a detailed cost estimate should be prepared with emphasis on component systems and following value engineering guidelines. Operation and maintenance costs should also be estimated based on the operation plan and maintenance schedule from Step 4. A cost-effectiveness analysis can then be prepared evaluating each site and establishing the priority for implementing construction.

Step 6 - Complete Design

The final step is to confirm that all objectives have been met. Several iterations back through previous steps may be needed to reach the most cost-effective solution. This is particularly true once site-specific costs for the various storage options have been developed.

OPERATION AND MAINTENANCE CONSIDERATIONS

The major operation and maintenance goal for detention storage facilities is to provide readily available, nuisance-free storage that will operate as designed whenever needed (even following prolonged periods of non-use). Such units should utilize as little unproductive space as possible while minimizing any visual impact.

Onsite

Onsite storage facilities should not require extensive maintenance after each use. Debris removal, care of the landscaping (if any), and inspection and maintenance of the outlet structure are all part of the routine operation of a storage facility.

Mosquito and algae problems can be eliminated by ensuring tht storage facilities drain completely and dry out between use. Storage ponds look best when a grass cover is kept on the basin slopes and floor. However, if the basin needs to be vegetation-free for any reason, visual screening can be provided by sight barriers such as trees.

Safety features must also be considered. Hazardous areas must be fenced to restrict access. Debris must be removed whenever it collects to prevent interference with the operation of the outlet structure and to eliminate hazards to users in a multipurpose facility.

In-System

Adequate information on the anticipated operation and effectiveness of the storage facility should be prepared. Experience has shown that frequent, periodic maintenance and equipment exercising is required. The nature of the atmosphere and conditions found where much of the equipment is located dictates this. Corrosion and clogging by debris are typical of the problems encountered by in-system facilities. The instrumentation needed to control the operation of dynamic regulators can be another source of problems. However, recent advances in the manufacture and design of instrumentation and controls have greatly reduced the problem.

COSTS

Construction costs for in-system storage have been reported for selected demonstration sites [5]. However, they are highly site specific. Adjusted to ENR 4000, the range of unit construction costs for in-system storage is from \$0.04 to \$0.94/gal (\$10.60 to \$221.90/m^3) of storage volume. Costs also vary considerably depending on the complexity of the flow regulators and control systems. For example, the cost of the control and monitoring system was 47% of the \$0.94/gal for that demonstration project [3].

Construction costs for offline storage for selected demonstration sites was reported to vary from \$0.16 to \$0.90/gal (\$42.30 to \$237.80/m^3) of storage volume [5].

Detailed operation and maintenance cost data are limited. No rule-of-thumb guides exist at present. Operation and maintenance costs must be estimated for specific facilities from the operation plan and maintenance schedule. For planning level studies, estimates can be developed from costs reported for operating facilities [5].

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Section 7

DESIGN OF SEDIMENTATION FACILITIES

The frequency of overflow operations to the total frequency of facility activations determines the design emphasis for downstream storage/sedimentation basins. By definition these facilities are intended to be end-of-the-pipe controls; discharges (overflows) are expected to be directly to receiving waters with or without further treatment. Storage/sedimentation is the most common and, perhaps, effectively practiced method of urban stormwater runoff control in terms of operating installations and length of service. Conversely, since it parallels historic sanitary engineering practice, storage/sedimentation is frequently criticized for lack of innovation due to its simplicity, and high cost due to size and structural requirements.

In this section, design considerations and procedures for downstream storage/sedimentation basins are presented and illustrated by example and through references to designed and operated facilities. Planning level costs and cost considerations are given.

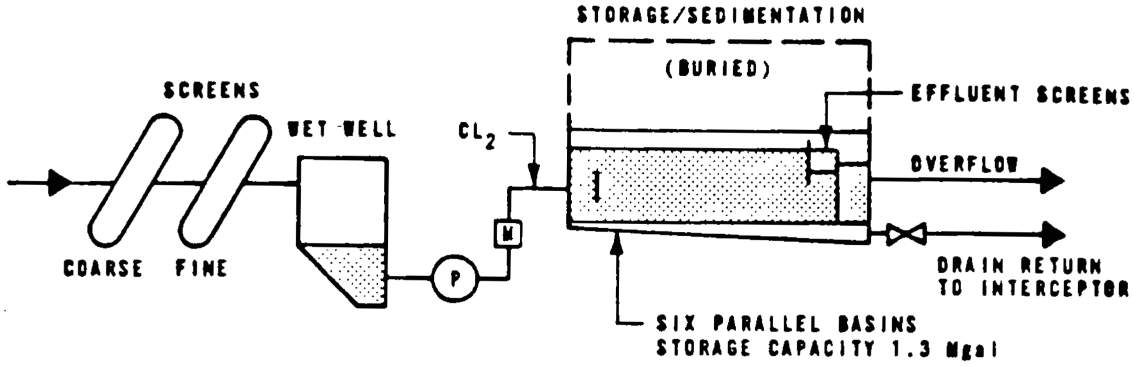
DESIGN CONSIDERATIONS

Functionally, the applications of downstream storage/sedimentation facilities vary from essentially total containment, experiencing only a few overflows per year, to flow-through treatment systems where total containment is the exception rather than the rule. For total containment, the major concerns are the usable storage volume, the provisions for dewatering, and post-storm cleanup. For flow-through treatment systems, performance hinges on treatment effectiveness and design considerations including loading rates, inlet and outlet controls, short circuiting, and sludge and scum removal systems.

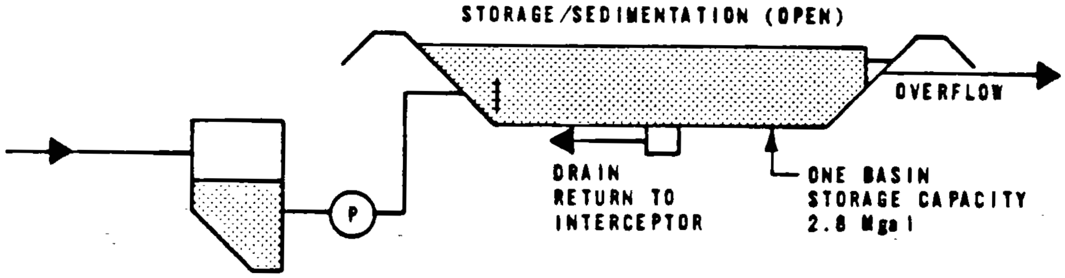
In the case of offline facilities, the option exists to selectively capture a portion of the stormflow (i.e., first flush contaminant load) and bypass the balance to avoid the loss of captured solids through turbulance and resuspension. Examples of representative CSO storage/sedimentation basins and auxiliary support facilities are shown in Figure 26.

Storage

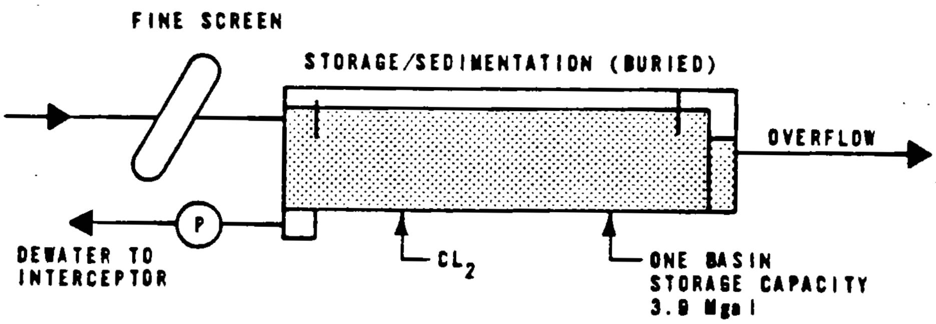
Required storage volumes to accomplish a specified level of control are best approximated through use of simplified continuous simulation models. A summary of representative models and their application is presented in the REQM Handbook [1]. Statistical methods recently developed by Hydroscience and by Howard have proved valuable for determining storage requirements for simple



BOSTON (COTTAGE FARM), MASSACHUSETTS

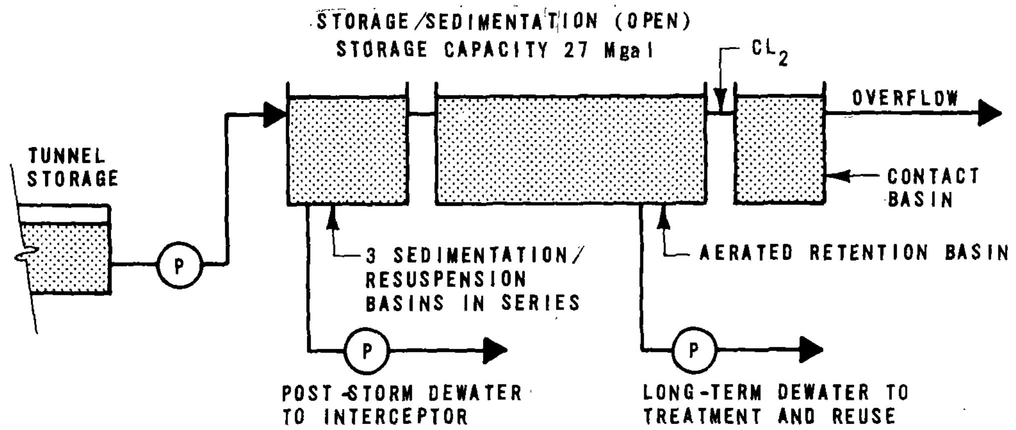


CHIPPEWA FALLS, WISCONSIN

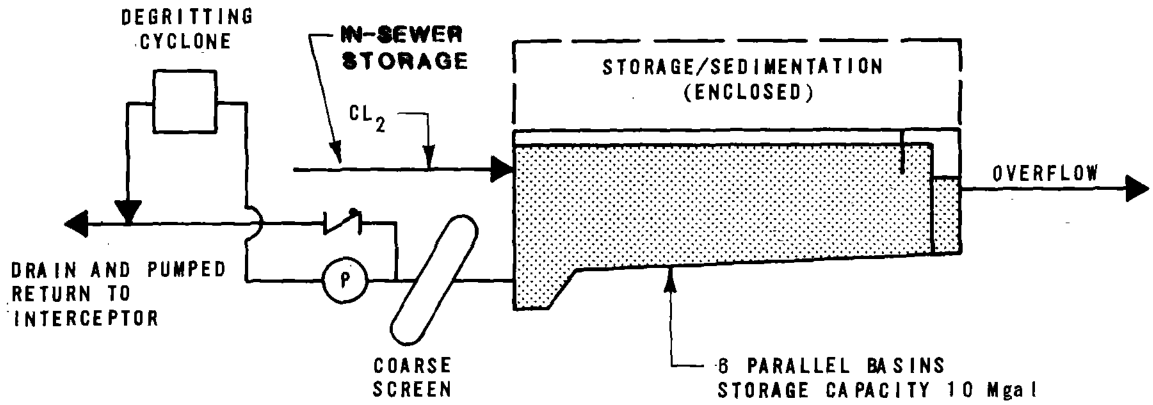


MILWAUKEE (HUMBOLT AVE.), WISCONSIN

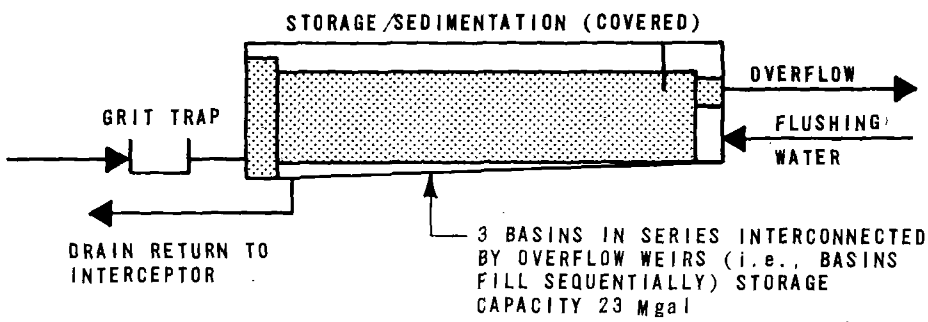
Figure 26. Representative CSO Storage/Sedimentation Basins and Auxiliary Support Facilities.



MOUNT CLEMENS

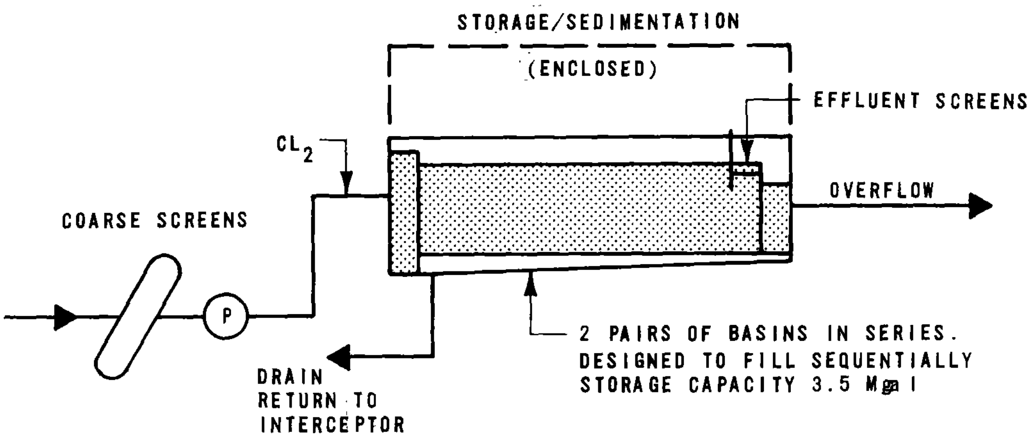


NEW YORK CITY (SPRING CREEK)

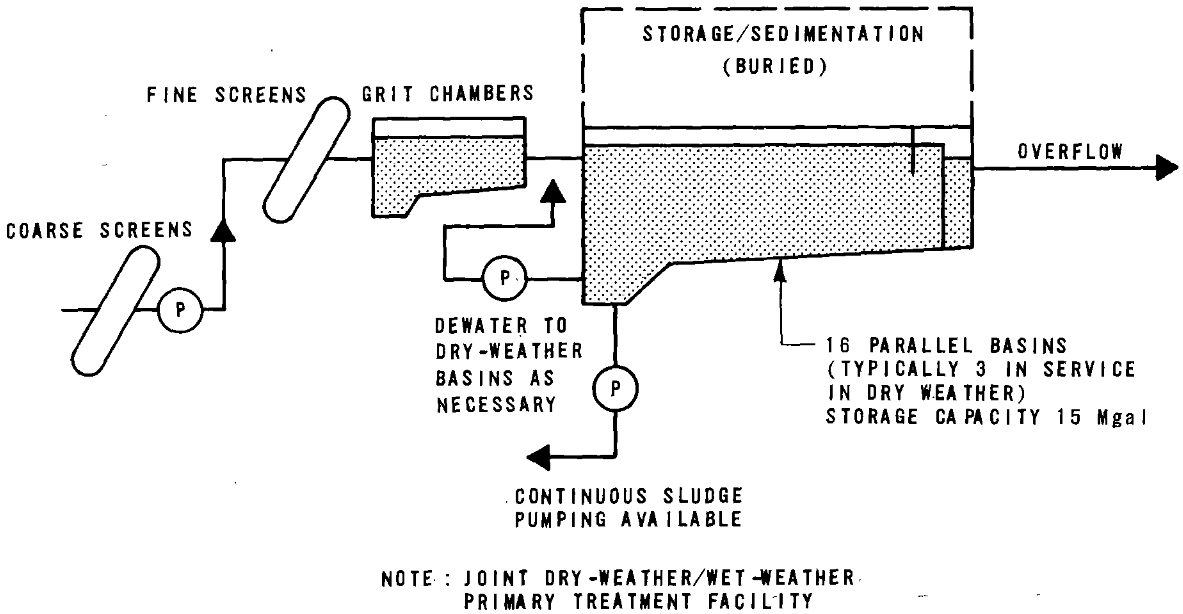


SACRAMENTO (PIONEER RESERVOIR)

Figure 26 (Continued)



SAGINAW (HANCOCK STREET)



SAN FRANCISCO (SOUTHWEST WPCP)

LEGEND

(P) PUMPS

CL_2 LOCATION OF CHLORINE OR HYPOCHLORITE ADDITION

Figure 26 (Concluded)

systems [2, 3]. Where the level of control objective is high (i.e., storage-treatment capacity is large compared to runoff volume) and urban development is intense, two particularly useful models are EPAMAC [4] and STORM [5]. The former is an extension of the Simplified Stormwater Model [6] and the areawide planning model ABMAC [7]. EPAMAC has been used as the base model for examples in this text. In cases where short-term flow dynamics are of major concern or where pervious areas of the watershed play a significant role, other, more detailed models such as SWMM [8] and NPS [9] may be required.

EPAMAC operates on an hourly timestep and is designed for applications on combined sewer systems. Inputs include hourly rainfall data, watershed subareas, runoff coefficients, runoff quantity and quality, routing time offsets, and network flow routing. At each network control node, the user specifies the available storage volume and dewatering (treatment) rate with its associated operating rules (i.e., pumps started and stopped as a function of filled storage volume for that timestep). When the hourly storage-treatment capacity is exceeded, an overflow (discharge) occurs.

The user selects trial storage volumes and associated dewatering (treatment) rates to fit the constraints of his system and through iterative analyses selects the combination that best satisfies his needs (i.e., overflow frequency, site limitations, and cost). Note that storage may be distributed over a series of nodes to represent upstream and inline storage options; however, only one network control node is allowed per run. For example, if storage is to be provided within a watershed through a sequential series of dispersed storage elements such as an upstream surface detention basin, an intermediate zone of inline storage, and a downstream storage/sedimentation basin, three sets of computational runs would be required. The first would consider only the upper watershed and its storage basin. The computed discharge from this basin would constitute a lateral infow for the second run. The second run would consider the additional tributary area to the inline storage facility plus the lateral inflow generated by the first run. In turn, the second run would produce a lateral inflow for the third run, which would reflect the storage benefits of both the upper surface basin and the inline storage in the intermediate zone. Finally, the third run would add any new tributary area flows to the lateral inflow generated by the second run and permit the sizing of the downstream basin.

Treatment Efficiency

Storage units may alter the wastewater characteristics of the applied stream by gravity separation. The suspended solids removal efficiency approaches a maximum under steady state, quiescent conditions. Unsteady, storm-induced flows generally produce velocity and, in some cases, temperature gradients in the sedimentation basins. These unsteady conditions may reduce the instantaneous suspended solids removal efficiency but, on an overall storm basis, may not drastically alter the overall removal efficiency. In its simplest sense, wastewater is made up of water containing particulates of varying specific gravities. When the convective forces transporting these particulates are reduced, those lighter than water start to rise and those heavier start to fall. This movement may increase collisions between particles and by adsorption or flocculation, large particles are formed that in turn furthers the separation. The movement continues until the particles settle to the floor of the chamber forming a sludge, rise to the surface forming a scum, or are carried out in the overflow.

Sedimentation theory and convective forces are lucidly described in wastewater engineering texts; however, there are two major problems: (1) wastewater tends to be quite heterogeneous with its makeup of heavier than, equal to, and lighter than water particulates changing from one moment to the next; and (2) the theory reflects idealized situations to which a myriad of modifiers must be applied to reflect real world conditions.

Design variables affecting hydraulic performance in general order of importance are surface-loading rates or overflow rates, detention time, basin geometry, inlet and outlet design, and rapid sludge removal. Potentially controllable parameters adversely affecting sedimentation performance are density currents due to temperature differentials between the incoming flow and the basin contents, density currents resulting from marine/estuarine water intrusion, turbulance generated by flow variations, and wind-induced currents. Generally, noncontrollable but important parameters of the raw wastewater are its suspended solids concentration, the effect of shear forces or velocities in the sewer on agglomerated organic particles, the proportion of settleable solids, and its age or septicity.

To estimate the efficiency of any sedimentation basin it is most important to know not only the suspended solids load but also the settleability characteristics and distribution of other pollutants associated with the solids. In other words, the particle size distribution, pollutants associated with the particles, and the density of the particles must be known. Therefore, detailed stormwater runoff and combined sewer overflow sampling is necessary to characterize the solids and pollutant distributions.

Samples should be flow weighted to produce a representative sample for analysis. The time variation in the solids loading is taken into account when flow weighted samples are used. Efforts should be made to ensure that the samples are representative with respect to depth within the flow stream.

A long documented but little used test to reflect these unique characteristics of a particular waste is the settling column test described by Metcalf & Eddy [10], Camp [11], and others [12, 13, 14, 15]. These tests provide a valuable aid in projecting storage/sedimentation performance in urban stormwater systems design.

Application and use of settling column tests are illustrated under the design procedure discussion to follow. Tests are normally run to record total suspended solids removal as a function of time and depth; however, it should be noted that settling column tests can be run on the basis of any quality parameter for which removals are accomplished by sedimentation. In association with a standard sieve analysis for particle size distributions, chemical analyses of the various solids fractions should be performed to determine the chemical and biochemical pollutants associated with the various particle sizes. This information can be used to estimate the effective removal efficiency for the various pollutants associated with the sediment. Biochemical analyses of the liquid portion should also be made so that the total pollutant load can be determined. The use of settling column test results along with application of the Storage/Transport Block of SWMM-Version III is recommended.

Typical removal efficiencies for total suspended solids as related to surface loading rates and detention times are shown in Figure 27. Each plot represents a "best fit" curve representation of a broad data scatter from a number of installations over a large number of events. For example, the empirical suspended solids removal efficiency, in terms of the surface loading rate for conventional primary treatment with mechanical sludge removal according to Smith [16] is:

$$R = 0.82e^{-S/2780} \tag{7-1}$$

where R = TSS removal efficiency, %
S = surface loading rate, $gal/ft^2 \cdot d$

The degree of scatter and limitations of theoretical approaches are illustrated in Figure 28, which is based on 24-hour influent and effluent samples from a primary treatment plant in San Francisco receiving storm and sanitary flows from a combined sewer system [17]. Thus, even a single plant will exhibit wide day-to-day fluxes in efficiency under the same surface loading rate. The potential for removal efficiencies to vary during individual storm events is shown in Table 18 [17], wherein performance over the first 2 storm hours (first flush) is compared to the average performance over the entire storm event.

Representative removal efficiencies associated with plain sedimentation of wastewater in conventional plants are listed in Table 19. Because of the limited data base, independent performance ranges cannot be presented for urban stormwater, but the presumption is that they will be similar. In San Francisco, comparison of heavy metals between filtered and nonfiltered stormwater samples indicated that the majority of heavy metals were associated with the solids fraction [17]; thus, effluent quality improvement would be expected to be associated with sedimentation. In the limited number of storm composites measured before and after treatment, however, a conclusive trend was not apparent.

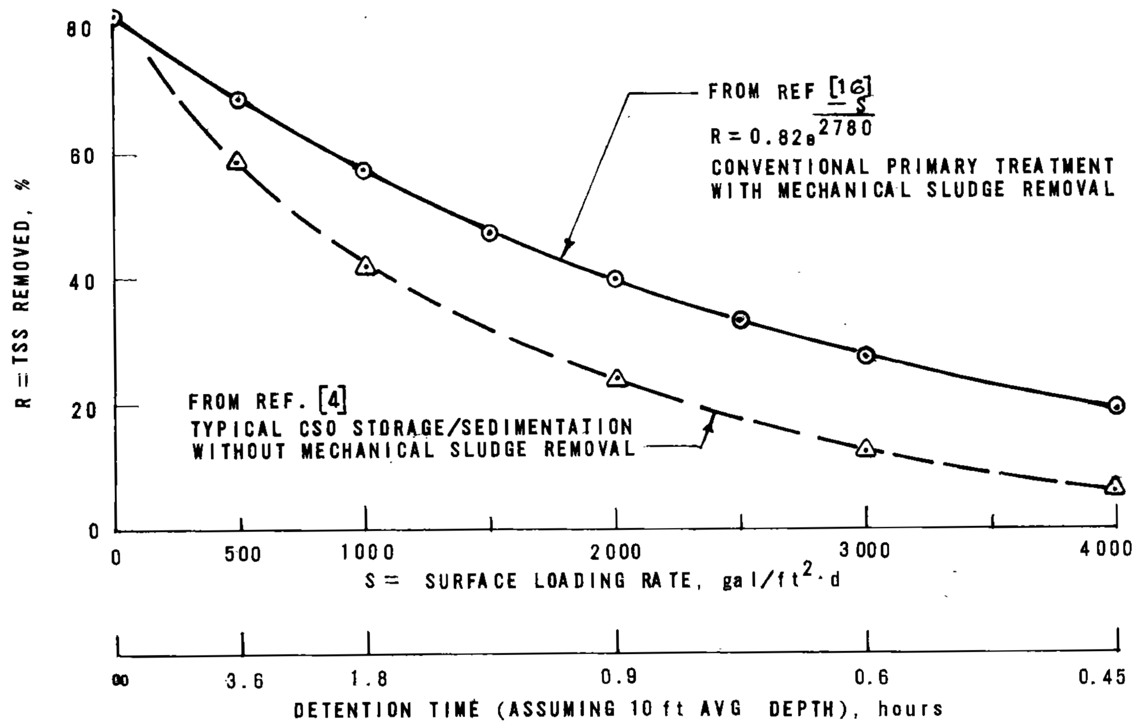


Figure 27. Typical TSS removal efficiencies by sedimentation

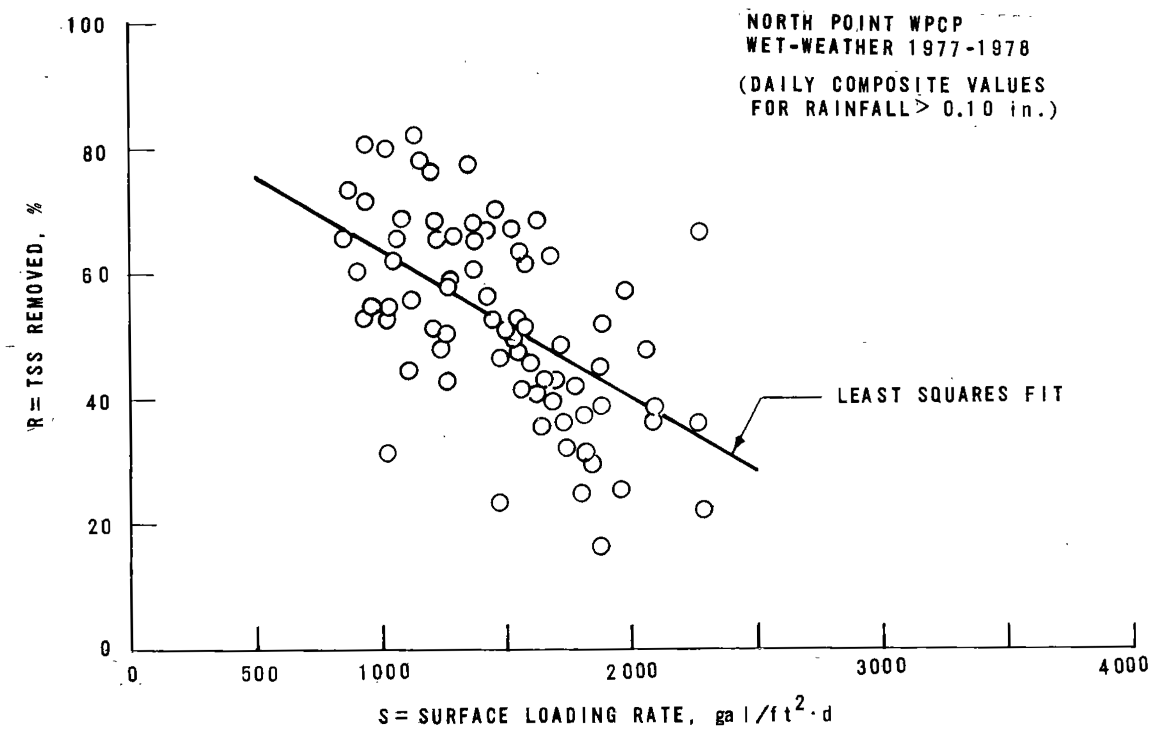


Figure 28. Experienced TSS removal efficiency variations [17]

Table 18. PILOT PLANT PERFORMANCE ON RICHMOND-SUNSET STORMWATER (CSO) FLOWS [17]

Date	Test duration h	Surface-loading rate, $gal/ft^2 \cdot d$	Total storm: Avg TSS Influent	Total storm: Avg TSS Effluent	Total storm: Avg removal, %	First flush[a]: Avg TSS Influent	First flush[a]: Avg TSS Effluent	First flush[a]: Avg removal, %
2/28/79	7.5	1,500-2,400	128	87	32	176	92	48
3/16/79	14.5	1,600-2,400	111	78	30	173	105	39
3/26/79								
a.m.[b]	22.5	2,000-2,400	98	49	50	255	118	61
p.m.	--		--	--	--	152	42	72

a. Average of all grab samples over first 2 hours of storm unless otherwise noted.
b. Morning shower lasted only 1 hour; main storm started 4-1/2 hours later.

Table 19. COMMON REMOVAL EFFICIENCIES ASSOCIATED WITH PRIMARY SEDIMENTATION OF SANITARY WASTEWATER [17]

Wastewater	Removal efficiency, %
BOD	25-40
TSS	40-70
Settleable solids	85->95
Bacteria	25-75
Total nitrogen	5-25
Total phosphorus	5-20
Grease and oil	40-60

Studies for Milwaukee have developed process curves for detention tanks, evaluating pollutant reduction and volumetric efficiency for several tank volumes. Suspended solids and BOD retention and percent of storm volume retained for both wet- and dry-year rainfalls are shown in Figure 29 [18]. The study also showed a decreasing efficiency per unit volume as tank size increases, as shown in Figure 30.

Disinfection

Where disinfection is often required in a storage/sedimentation basin, a minimum contact period is specified. Further, the consumption of disinfectant (typically chlorine or a chlorine derivative) and its effectiveness are adversely impacted by solids in the flow. Therefore, where detention periods dictated by storage requirements are significantly longer than the contact period required, multistaged basins should be considered with the disinfectant added to the final stage(s) only. In this manner, the benefits of the partially clarified wastewater will be realized. Common dosage requirements are 15 to 30 mg/L of available chlorine.

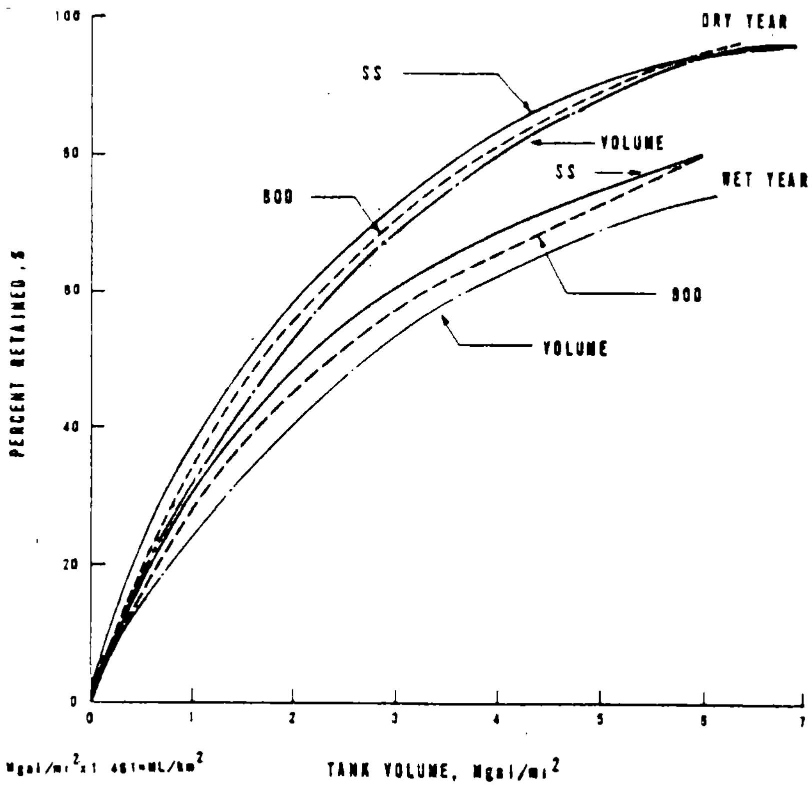


Figure 29. Pollution and volumetric retention versus storage tank volume for wet- and dry-years.

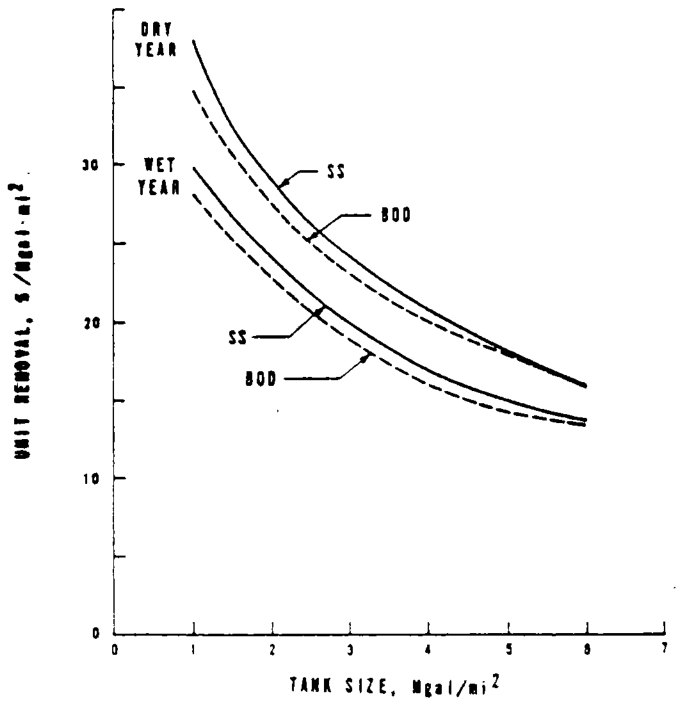


Figure 30. Unit removal efficiencies for combined sewer overflow detention tanks

High-rate disinfection of raw and prescreened combined sewer overflows using sodium hypochlorite with high velocity gradients in the contact chamber or using chlorine alone or chlorine followed by chlorine dioxide has been tested [19, 20, 21, 22]. Results equivalent to that of normal practice were achieved using dosages as low as 8 mg/L and contact times as short as 3 minutes and less. Flow control must be established and the flowrates known in order to effectively pace the dosage. Because of the rapid changes in flow typically received in storage/sedimentation basins, pacing disinfection additions solely by effluent residual monitoring has not been effective [23].

Site Constraints

Whereas approximately one out of ten conventional wastewater treatment plants is covered, the reverse is the general rule for downstream storage/ sedimentation basins serving combined sewer areas. This is because available land along waterfronts within the urban core is typically in very highly developed or recreation oriented areas; thus, in the public's mind at least, it is incompatible with open raw sewage basins. Historically, treatment plants have been built in quasi-isolated areas and development has encroached on the sites. Conversely, CSO systems and their associated urban development exist, and it is the treatment facility that must do the encroaching. In smaller communities (i.e., Chippewa Falls, Wisconsin [23], and Mt. Clemens, Michigan [24], comparatively isolated centrally located areas have been found and open basins constructed and operated without reported nuisance. In the majority of cases, however (i.e., Akron [25], Boston [26], Milwaukee [18], New York [27], Sacramento, Saginaw [28], and San Francisco), the facilities are covered and in some cases buried. In the cases of Akron and Saginaw, use is made of the land above the structure.

In San Francisco, problems with limitations in usable waterfront space were coupled with the need to intercept a multitude of dispersed overflow points in arriving at an innovative storage-transport concept. In this case, large, elongated downstream storage/sedimentation basins--super sewers--were constructed that combined storage/sedimentation functions (i.e., pretreatment for overflows) with interception and transport functions. The North Shore consolidation project, for example, provides a monolithic box conduit and tunnel structure snaking along 3 miles of waterfront. The project intercepts seven overflow points, provides 23 Mgal (87 m^3) of storage, conveys flows to a central location for pumping to treatment, and provides pretreatment of overflows (an average of four per year) by sedimentation and skimming. The fact that "super sewers" can provide significant treatment during periods of overflow is demonstrated in operating data taken from the 2 mile long, 62 Mgal (235 m^3) capacity, Red Run Drain near Detroit [29] and shown in Table 20.

Even covered facilities vary in that some permit operator access during operations (i.e., walkways and working space above the free water surface), where others can be entered (as in sewers) only in a dewatered condition. The net impact is frequently a doubling and redoubling of the basic functional cost; however, the alternatives are typically no more acceptable than open sewers would be.

Table 20. PERFORMANCE OF THE RED RUN CSO SEDIMENTATION/TRANSPORT BASIN [29]

	Total suspended solids			Volatile suspended solids		
Storm date	Influent, mg/L	Effluent, mg/L	Removal, %	Influent, mg/L	Effluent, mg/L	Removal, %
3/14-15/78	116	102	12	62	36	42
3/21-22/78	52	36	31	32	20	38
5/8/78	238	168	29	128	54	59
5/13/78	114	38	67	64	4	94
5/30/78	294	152	48	170	26	85

Limited studies of odor generation from stored urban stormwater (CSO) conducted at San Francisco [20] for two storms. These studies showed that for 1,500 gallon (5,678 L) samples of first flush stormwater stored in covered but vented storage, hydrogen sulfide generation peaked 24 hours after collection but that concentrations had not reached a distinguishable level (0.3 ppm-volumetric), even after 48 hours. Highest odor potential should be expected during dewatering operations as settled sludge is exposed. Options for sludge removal systems, performance assessment under variable flowrates, and other design details are discussed in the following section.

DESIGN PROCEDURE/EXAMPLE

A suggested design methodology was shown previously in Section 4. Each of the indicated steps is discussed below and examples are introduced where applicable.

Step 1 - Identify Functional Requirements

As noted previously, the intended operational function of the downstream storage/sedimentation basin will determine its design emphasis (i.e., does its treatment function rank primary or secondary with respect to storage). Obviously, a facility that will spill or discharge under all but the smallest of storms (i.e., a typical detention-chlorination facility) should be designed as a treatment facility. For a discharge frequency of a few times a year, a design predicated mainly on construction and operation economics would be warranted. Area hydrology, system hydraulics, and overflow frequency objectives will determine the volumetric capacity required. As stated earlier, simplified continuous simulation models are generally best suited to this task and detailed user manuals [1, 4] have been prepared.

Answers to be provided by the model or developed from the model output include:

- Design volumetric capacity or matrix of storage-dewatering (treatment) rate combinations that meet overflow frequency criteria.

- Frequency distribution of unit operations by month, year, and period of record.

- Frequency distribution of operation durations, storage volumes utilized, treatment rates experienced, overflow events, overflow rates, overflow durations, and between storm downtime availability.

- First cut assessment of solids applied, solids retained or diverted, and solids overflowed.

- First cut assessment of the impact of the reduced solids and pollutant load on the receiving water quality and the determination of the cost-effectiveness of the proposed facilities.

Where an NPDES permit has been issued, it must be consulted to identify any restrictions on discharges in terms of concentrations, mass loadings (i.e., basin plan waste load allocations), disinfection requirements, and reporting criteria. Where NPDES permits have not been issued, target criteria must be established through meetings with regulatory agencies having jurisdiction, and must be supported through cost-effectiveness analysis.

Step 2 - Identify Site Constraints

Sites for downstream storage/sedimentation basins should be identified and cataloged with respect to at least the following criteria:

- Accessibility to the collection conduit, the interceptor (for postevent dewatering), and a suitable overflow point for discharges.

- Total usable area and its dimensions and configuration.

- Hydraulic data on influent levels, receiving water levels, (normal and flood), and interceptor levels to identify stage and pumping requirements for the proposed facility.

- Environmental setting such as proximity to residences, public facilities, compatible and noncompatible land uses, visual exposure, and prevailing winds.

- Geotechnical conditions and probable structural requirements (i.e., pile supports, hazards to adjacent structures and utilities, etc.)

- Accessibility to utility services and for construction and operation activities.

Typically, this information will be used in selecting the main treatment geometry in Step 4.

Step 3 - Establish Basis of Design

The purpose of this step is to determine influent characteristics and loading rates necessary to meet the requirements set down in Step 1. Representative influent characteristics may be developed, in the case of CSO systems, from direct field measurements and supplemented by an analysis of dry-weather wastewater treatment plant influent data during wet-weather operations. These data should be segregated by (1) storm size (rainfall recorded), (2) seasonal occurrence, (3) time into the event, etc.

In addition, it is recommended that settling column tests be performed as a basis for predicting basin performance. These tests have found wide acceptance in the industrial waste treatment field (where designers freely admit lack of knowledge of a particular wastes settling behavior). However, these tests are rarely performed on municipal wastewaters where (1) the assumption is made that behavior will be typical, or (2) that the sedimentation unit process will be followed by additional unit processes; thus the relative importance of primary settling characteristics is small. In urban stormwater management, the additional testing is certainly warranted. In addition to knowing the influent suspended solids concentration, it would be extremely informative to know the range of settling velocities for the particles and the mass that can be settled within a reasonable time period when selecting surface loading rates and detention times for design.

In translating the idealized (quiescent) settling column results in design, texts caution that to account for the less than optimum conditions encountered in the field, the design settling velocity or surface loading rate obtained from column studies should be multiplied by a factor of 0.50 to 0.85 and detention times by a factor of 1.25 to 2.0 [10, 23]. Heinke et al. [30] found good correlation between settling column results and measured performance of three municipal plants in Canada monitored over a 4-year period.

Predictions of sedimentation tank performance made from settling column tests compared closely with actual tank performance under low overflow rates of about 600 gal/ft^2·d (24 m^3/m^2·d). For higher overflow rates, the actual performance of the settling tanks was much better than the predictions from the settling tests.

The use of the suspended solids removal efficiencies for various overflow rates can be used to predict the efficiency of a sedimentation basin for unsteady flow conditions. A time-step approach utilizing the overflow rate, and the predicted efficiency at that overflow rate, and the influent suspended solids concentrations can estimate the overall efficiency for a storm or series of storms.

Example 1 illustrates the use of settling column test results in establishing a projected performance curve.

Example 1. COMPUTE TSS PERFORMANCE CURVE FROM SETTLING COLUMN TEST RESULTS

Specified Conditions

1. Laboratory test results were obtained from a settling column test of a 2 hour composite sample of "first flush" CSO. Test column was 6 in. diameter, 10 ft high, with sample taps at 24 in. centers. The composite sample was premixed and pumped into the column. Samples were drawn from each tap initially and repeated at specified time intervals. TSS results for the individual samples were as follows:

TSS results, mg/L

Initial depth, in.	Elapsed time, minutes				
	0	30	60	90	120
5	202	136	112	96	--
29	240	172	122	126	110
53	384	148	126	132	118
77	384	236	140	142	124
101	408	226	154	138	118
Mean values	324	184	131	127	118

2. Sedimentation tank depth is 10 ft.

Assumptions

1. Assume a surface loading rate scale factor of 0.75 to translate the "idealized" column results to a field basin.

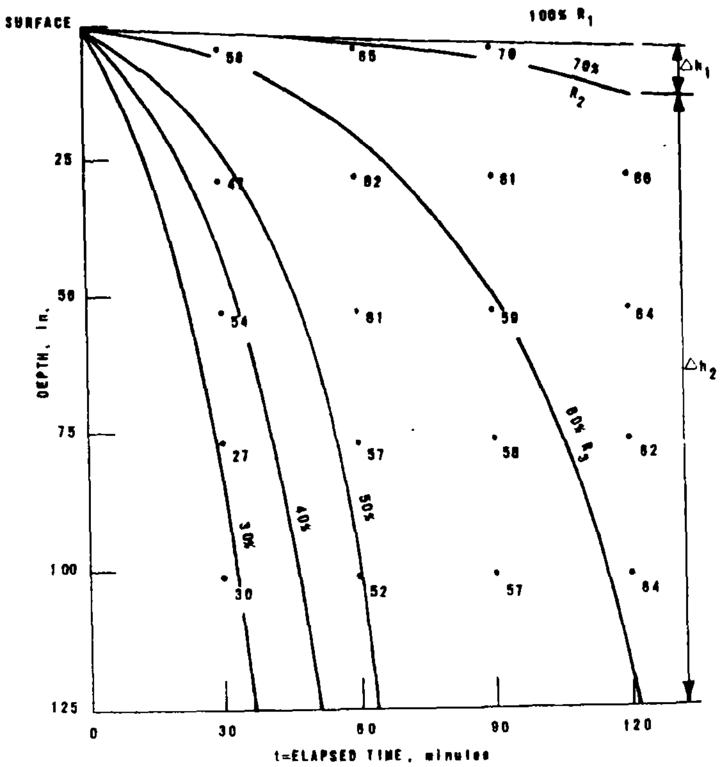
Solution

1. Calculate TSS removals as a percent of initial mean concentration.

TSS removal results, %

Initial depth, in.	Elapsed time, minutes				
	0	30	60	90	120
5	--	58	65	70	--
29	--	47	62	61	66
53	--	54	61	59	64
77	--	27	57	56	62
101	--	30	52	57	64

2. Plot results to scale and sketch in best fit removal curves for 30%, 40%, 50%, 60%, and 70%.



3. Compute the percent removal at 30, 60, 90, and 120 minutes by proportionality. For example, at t = 120 minutes, percent removal

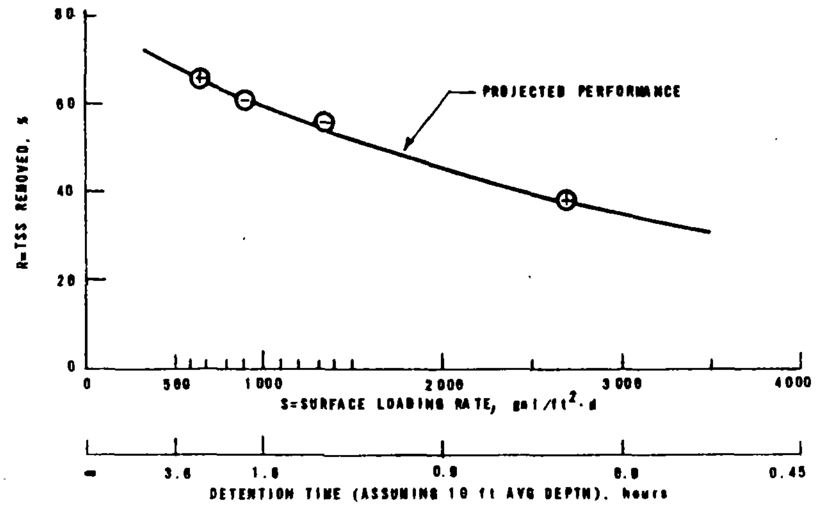
$$= \frac{\Delta h_1}{\Delta h_1 + \Delta h_2} \times \frac{R_1 + R_2}{2} + \frac{\Delta h_2}{\Delta h_1 + \Delta h_2} \times \frac{R_2 + R_3}{2}$$

Time	Calculation	Removal
120	$\left(\frac{9}{120} \times \frac{170}{2}\right)+\left(\frac{111}{120} \times \frac{130}{2}\right) = 6.37 + 60.13$	= 66.5
90	$\left(\frac{4}{121} \times \frac{170}{2}\right)+\left(\frac{42}{121} \times \frac{130}{2}\right)+\left(\frac{75}{121} \times \frac{116}{2}\right)$	= 61.3
60	$\left(\frac{1}{122} \times \frac{170}{2}\right)+\left(\frac{16}{122} \times \frac{130}{2}\right)+\left(\frac{85}{122} \times \frac{110}{2}\right)+\left(\frac{20}{122} \times \frac{98}{2}\right)$	= 55.5
30	$\left(\frac{4}{123} \times \frac{130}{2}\right)+\left(\frac{17}{123} \times \frac{110}{2}\right)+\left(\frac{20}{123} \times \frac{90}{2}\right)+\left(\frac{42}{123} \times \frac{70}{2}\right)+\left(\frac{40}{123} \times \frac{55}{2}\right)$ =	37.9

4. Using 10 ft depth, compute surface loadings corresponding to detention times and apply scale factors (0.75 surface loading and 1.33 to detention time) for projected prototype performance.

Unscaled (ideal) performance			Projected performance		
Detention time, min	Surface loading, gal/ft^2·d	Removal efficiency, %	Detention time, min.	Surface loading, gal/ft^2·d	Removal efficiency, %
30	3,591	38	40	2,693	38
60	1,796	56	80	1,347	56
90	1,197	61	120	898	61
120	898	66	160	674	66

5. Plot projected performance results for use in Example 7..



Comments

First flush test behavior shows continued good performance at high overflow rates. Problem in completely mixing the sample in the settling column at t = 0 is evident in the test results. Ideally, the concentrations at each depth at t = 0 should fall within 10% of the mean value.

Step 4 - Select Main Treatment Geometry

The geometry of a downstream storage/sedimentation basin will be governed by the constraints identified in Step 2, the loading rates selected from Step 3, and the operational concept developed from the Step 1 frequency analyses. For example, if a large number of the total plant operations will use, say 50% of the storage capacity or less, a compartmented basin with sequential filling could greatly reduce cleanup operations without any impact on performance. Also, if the characterization data indicate a pronounced first flush, segregating this load from the balance of the storm may be beneficial from both a cleanup and performance aspect.

Basic reasons for dividing storage/sedimentation basins into compartments are:

- To reduce short circuiting
- To facilitate cleanup and sludge removal from tanks that fill in series
- To permit isolation of slug loads in individual tanks
- To provide operational redundancy through parallel units

When compartments are linked in series (Saginaw, Sacramento), short circuiting is minimized. When compartments are operated in parallel (Boston, New York City), longitudinal flow (resuspension) velocities are minimized. Camp [11] notes that it has been common practice to limit design longitudinal velocities in settling tanks to about 3 ft/min (0.02 m/s). However, he notes that despite a dearth of experimental data there is increasing evidence that higher velocities are accompanied by better removals. This phenomenon occurs because the additional flocculation caused by turbulence in the tank may speed up the settling to a greater extent than turbulent mixing retards it. Heinke et. al [30] suggest 8 ft/min (0.04 m/s) as a maximum design value for primary sedimentation tanks based on field observations. Initial results from Saginaw [24], as shown in Table 21, suggest potential benefits from the series (two-stage settling) configuration. However, at present there are no settling column test results to compare the theoretical and actual removal efficiencies based on influent characteristics and basin design. One possible explanation is that CSO contains not only those solids found in sanitary sewage but also additional grit and sand resuspended from the sewer or flushed into the sewer from urban areas.

Table 21. PERFORMANCE OF THE HANCOCK STREET SEDIMENTATION, SAGINAW, MI

Storm date	Avg surface loading rate, gal/ft^2·d	Longitudinal velocity ft/min	Suspended solids Influent, mg/L	Suspended solids Effluent, mg/L	Suspended solids Removal, %
8/19/78	970	3.7	896	62	93
9/13/78	1,235	4.7	149	27	82
9/20/78	2,270	8.7	420	232	45

When treatment effectiveness is the primary concern, inlet and outlet works should be designed as in conventional wastewater treatment practice [10, 31], (i.e., to minimize density currents, short circuiting, resuspension, and turbulence). The inlet works should spread the influent evenly across the vertical cross-section of the tank without resuspending the sludge blanket. Effluent weir loading rates of 10,000 to 40,000 gal/ft·d (125 to 500 m^3/m·d) are representative of conventional design [10]. When the basin's function is basically storage, inlet and outlet works should be as simple as practical, but effluent and, probably, interstage baffles should be provided to minimize scum carryover.

Two major, and frequently the most controversial, design decisions will be the degree and means of covering the basins and the means of solids and floatables removal. Both are expected to impact cost and aesthetics more than they do performance; however, past practices may have underestimated the value of continuous sludge removal. Of the 10 downstream storage/sedimentation basins reviewed in a recent state-of-the-art assessment [32], all contemplated a batch (fill-operated-drain) operation; all but two were covered; and none provided for solids removal until after the event. A potential liability of allowing solids to accumulate in the basin is the resuspension of those solids during another operation of the basin before the solids can be removed. Discharge of any of these solids in the overflow results in an apparent reduction of the basin efficiency. Data from New York City's Spring Creek Facility [33], the Cottage Farm facility in Boston [32], and the Whittier Street facility in Columbus [32] exhibit this behavior, especially under surface loading rates exceeding 3,000 gal/ft^2·d [37.5 m^3/m· d].

More important, perhaps, in the design and performance assessment of facilities such as those in New York at Spring Creek (see Table 22) and Boston that provide both storage and treatment (ignoring for the present their primary function for overflow disinfection), are the storm events totally or substantially contained. As an illustration, Table 23 reflects the total facility performance at Spring Creek when totally contained events are credited as 100% removal. Obviously, this illustration could be expanded to account for the efficiency of the downstream plant and flows retained in the basin and subsequently returned. However, the greater the percentage of events totally contained, the less will be the impact on net performance of the short-term efficiencies or inefficiencies during discharge.

Table 22. PERFORMANCE OF THE SPRING CREEK
AUXILIARY WATER POLLUTION CONTROL PLANT [33]

Year	No. plant operations		Events totally contained, %	Parameter	Monthly averages		
	Startups	Discharges			Influent, mg/L	Effluent, mg/L	Removal, %[a]
1977	110	24	78	TSS	166	103	38
				BOD	79	50	37
1978	52	27	48	TSS	162	71	56
				BOD	56	31	45

a. Removal efficiencies reflect periods of discharge only.
b. Months where average effluent concentrations exceed average influent concentration, excluding zero discharge months.

Table 23. NET BENEFITS APPROXIMATION OF SPRING CREEK FACILITIES

Year	Parameter	Calculation	Net removal efficiency, %
1977	TSS	$\frac{86 \times 100 + 24 \times 38}{110} =$	86.5
	BOD	$\frac{86 \times 100 + 24 \times 37}{110} =$	86.2
1978	TSS	$\frac{25 \times 100 + 27 \times 56}{52} =$	77.2
	BOD	$\frac{25 \times 100 + 27 \times 45}{52} =$	71.4

Example 2 illustrates the use of continuous simulation model (EPAMAC) results and a projected performance curve for assessing the overall treatment effectiveness of two operational concepts:

- Concept 1 Multicompartmented basin with all units committed for each event.
- Concept 2 Same facilities as Concept 1, but with limiting number of compartments online to approach but not exceed a maximum overflow rate objective.

Covering downstream storage/sedimentation basins on CSO systems is frequently a design requirement for environmental compatibility. Design considerations include cost, equipment access, and the creation of a potentially hazardous and corrosive environment. Adamski [34] sums up an operator's perspective of New York City's experience in covering wastewater treatment plants with the conclusion that

> "...covered treatment plants are difficult and costly to build and operate. That even by improving the design features, certain difficulties cannot be overcome. The reason for covering is usually a result of uneducated planners, architects, and citizens who impose a burden on the operator because the choices open to them are limited. When evaluating the need for a roof on a sewage treatment plant, the reason for it must be clearly defined and remain clear in the course of review and the rhetoric of protest. If the need is for odor control, then the best solution, whether operating or structural, should be selected after adequate study of alternatives. If the need is aesthetic, then the point of viewing must be kept in mind (whether from the ground or from above). If the need is land or recreational opportunity, then that should be explored. In all cases, the cost of satisfying these needs should be spelled out and the ability to choose other needs to satisfy. Also, the operator should be considered so that his job can be made easier." [34]

EXAMPLE 2. COMPARE TREATMENT EFFECTIVENESS OF TWO ALTERNATIVE OPERATIONAL CONCEPTS: (1) MULTICOMPARTMENTED BASIN WITH ALL AVAILABLE BASIN CAPACITY ONLINE, AND (2) SAME FACILITIES BUT LIMITING NUMBER OF COMPARTMENTS ONLINE TO APPROACH BUT NOT EXCEED MAXIMUM SURFACE LOADING RATE OBJECTIVE.

Specified Conditions

1. Maximum surface loading rate objective is 3,000 gal/ft^2·d.
2. Average annual operating requirements are as follows (from EPAMAC system analysis):

Flowrate, Mgal/d	Average annual hours of operation
>400	0
400	260
320	25
240	38
160	89
80	221
Total	633

Total volume captured and treated by sedimentation - 4,940 Mgal
Total TSS applied - 6,923,000 lb

3. The storage/sedimentation facility is to have five parallel, identical basins with average sidewater depth of 10 feet.

Assumptions

1. For purposes of comparing options, assume TSS influent concentration is constant.
2. Performance curve developed in Example 1 applies.

Solution

1. Compute mean TSS concentration applied.

$$\frac{6{,}933{,}000 \text{ lb}}{4{,}940 \text{ Mgal}} \times \frac{1 \text{ mg/L}}{8.34 \text{ lb/Mgal}} = 168 \text{ mg/L}$$

2. Compute required surface area for facility based on maximum surface loading rate.

$$\frac{400 \text{ Mgal/d}}{3{,}000 \text{ gal/ft}^2\cdot\text{d}} = 133{,}333 \text{ ft}^2$$

3. For Concept 1, compute surface loading rates corresponding to design flows (note in Concept 2 surface loading rate is always 3,000 gal/ft^2·d by definition).

   Read removal efficiencies from performance curve (Example 1) for each of these rates.

Flow, Mgal/d	Surface loading rate, gal/ft^2·d	Removal efficiency, %
400	3,000	35
320	2,400	41
240	1,800	48
160	1,200	56
80	600	67

2. Compute removal effectiveness of each

Concept 1

(80 Mgal/d)/24 x 8.34 x 168 x 221 h x 0.67 = 0.69 x 10^6 lb

(160 Mgal/d)/24 x 8.34 x 168 x 89 h x 0.56 = 0.47 x 10^6 lb

(240 Mgal/d)/24 x 8.34 x 168 x 38 h x 0.48 = 0.26 x 10^6 lb

(320 Mgal/d)/24 x 8.34 x 168 x 25 h x 0.41 = 0.19 x 10^6 lb

(400 Mgal/d)/24 x 8.34 x 168 x 260 h x 0.35 = 2.12 x 10^6 lb

Total removed = 3.73 x 10^6 lb
Net % removed = 54%

Concept 2

Total TSS applied 6.923 x 10^6 lb x 0.35 = total removed = 2.42 x 10^6 lb.

5. Compute net effectivess improvement of Concept 1 over Concept 2.

(3.73 - 2.42)/2.42 = 54% improvement in annual TSS removal by adopting Concept 1 over Concept 2

Comment

For the conditions stated, it is apparent that Concept 1 (maximizing tankage online) is associated with significant benefits. This might not be the case where the adopted maximum surface loading is much more conservative, where the performance change as a function of surface loading rate is less pronounced, or where the required operating history is significantly different.

Typically, where flushing is the adopted system (whether through fixed or movable nozzles), a center dewatering trough--invert slope approximately 1%--is provided running the length of the basin with floor side slopes to the trough at 5 to 10%. Basins or individual bays range from 27 ft (8 m) to 80 ft (24 m) in width, and typically 10 to 20 ft (3 to 6 m) in sidewater depth. Boston provides manual cleanup after dewatering using fire hoses; New York uses traveling bridge mounted sprays; and Saginaw uses a combination of wall-mounted fixed sprays and strategically positioned high pressure fire nozzle stations.

Under the latter case, approximately 5 Mgal (19,000 m^3) of washwater (strained river water) is used per washdown cycle for the 23 Mgal (87 m^3) capacity reservoir [35].

Cited advantages of flushing water systems include low cost, thorough cleaning performance, and minimum of mechanical equipment exposed to the corrosive environment. Principal disadvantages would appear to be the inability to remove sludge in other than a dewatered basin condition, energy requirements for pressurization, and the increased liquid volume to be treated through the pump-back system.

In lieu of flushing, Milwaukee uses seven mechanical mixers to resuspend solids from its 0.7 acre (0.3 ha) floor area during dewatering operations; New York City uses a series of hydraulic nozzles on a traveling bridge to resuspend solids; Columbus uses a traveling bridge mounted scraper blade; and San Francisco proposes to use conventional chain and flight collectors in its Southwest wet-weather primary treatment plant. In the latter case, virtually all operations (averaging 633 operating hours per year) will be in a flow-through treatment mode as the principal storage is provided elsewhere in the system. To avoid problems experienced in earlier stormwater demonstration projects where chains and drives have significantly corroded (rust bound) under the normal fill and draw operations, nonmetallic chains are under consideration. It is also noted that with the capability for continuous sludge removal, tanks may not have to be dewatered through most of the wet-weather season, easing maintenance requirements and maintaining a short response time (readiness-to-serve) for system activation. In Mt. Clemens, still another system will use air and water jets from wall mounted headers to resuspend solids in a slurry as a modification of the more typical flushing system [24].

One means currently used in Europe for removal of settled solids is a submersible pump suspended from a traveling bridge. The depth of the pump is automatically controlled so that sludge of the proper consistency is withdrawn (without disturbing the pond bottom when unlined earthen ponds are used).

At Chippewa Falls, solids are removed from the dewatered basin by mechanical equipment (street sweepers, loaders, and dump trucks). The basin is lined with asphalt and has a ramp down the side for vehicle access. A similar system is used at several facilities in Europe also.

Step 5 - Identify and Select Pretreatment Components

Pretreatment components are selected on the basis of enhancing performance and/or operations. Typically, the choices include coarse screening and grit removal. The units, if provided, may be located at the facility or upstream of pumps serving the facility. In practice, the adopted components include none, some, or all of the above. The purpose of the coarse bar racks with 2 to 4 in. (5 to 10 cm) clear openings is to remove heavy objects of all descriptions from the flow to protect downstream equipment and to prevent travel of objects to more inaccessable locations. Finer screens with 0.75 to 1.5 in. (2 to 4 cm) clear openings typically remove rags and finer solids that tend to clog process piping, valves, and pumps. They also trap many of the floatables that otherwise might appear in the effluent. Where basin overflows are rare, either or both have been omitted. Separate grit removal normally would be required only where treatment is the primary role of the facility and where grit is to be handled separately from the sludge. In some facilities, grit is removed from the sludge after sedimentation by using cyclone grit separators. Flow measurement and recording is recommended for all facilities that discharge frequently; whereas stage measurement and recording should suffice for basins which seldom overflow. Flow measurement is essential for pacing disinfectant dosages to reduce the chance for ineffective application and toxic carryovers.

Step 6 - Detail Auxiliary Systems

Auxiliary systems, those which support and complete the primary function, typically include sludge removal and processing, flushing, disinfection, air handling and odor control, energy (power, lighting, heating), and instrumentation and control. Sludge processing considerations include (1) the location and method of ultimate processing, (2) method of transport, (3) the impact on existing facilities, and (4) constraints (i.e., pumping rates, solids concentration, pretreatment) that must be observed. The simplest and most practiced solution is to return the sludge to the interceptor, sometimes with an intermediate degritting step.

Flushing water system evaluation includes source of supply, quantity and rate of application, pressure requirements (typically up to 150 $lb/in.^2$ or 11 kg/cm^2), distribution system, and method of control. A conceptual drawing of the Sacramento system as adapted to San Francisco is shown in Figure 31 [36]. The design flushing water application rate is 30 gal/min ft of basin length (21 L/m s) with 100 ft (30 m) segments to be flushed sequentially.

When facilities are covered, air handling and gas monitoring (for explosive, corrosive, and toxic potential) are important considerations. In selecting air change requirements for enclosed secondary treatment plants, Adamski [34] notes that two air changes per hour proved inadequate to control misting and that later New York City designs provided for a minimum of six air changes per hour. Common practice in covered CSO storage/sedimentation basins seems to be 6 to 12 air changes per hour with the higher figures based on a full liquid depth condition. Variable rate air handling control through staging or speed control appears desirable for energy conservation. Standard practices for odor control should be evaluated [37].

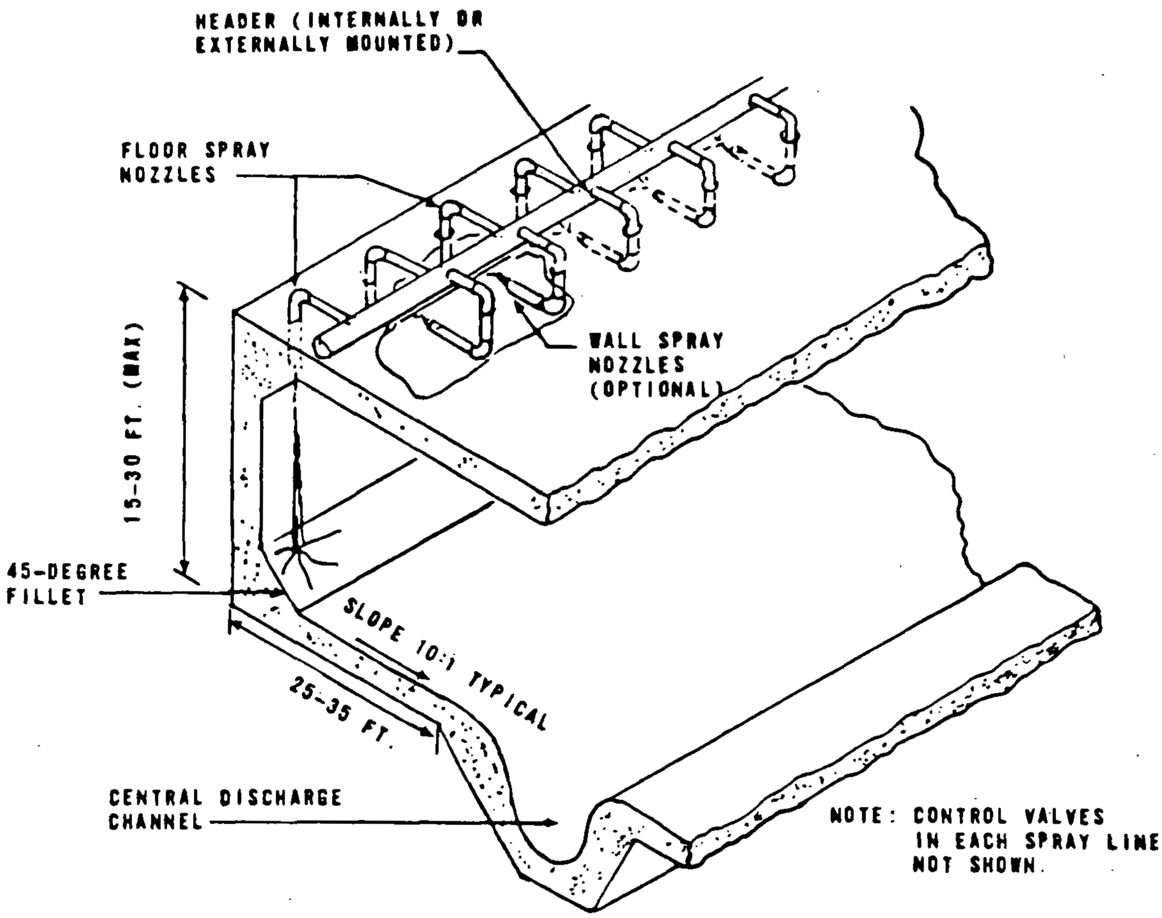


Figure 31. Flushing water system concept [36].

As a rule, instrumentation and control systems should be as simple as possible and should be designed giving full recognition to the corrosive environment and the level of operation and maintenance to be provided. Areas of study recommended are:

- Status and performance monitoring
- System activation
- System deactivation
- Auxiliary system control

Step 7 - Estimate Costs and Cost Sensitivities

Detailed cost estimates should be prepared with emphasis on component systems and following the value engineering guidelines. For example, what is the base cost of the facility to provide the functional requirements identified in Step 1 on the site selected in Step 2? What was the added cost of covering including air handling? What was the added cost of pretreatment? sludge removal? instrumentation and control? How much do site conditions impact the base cost? This analysis should lead to a more cost effective total design.

An operations plan and staffing and maintenance schedule should be finalized at this point and operations and maintenance cost projections made. As noted in earlier state-of-the art assessments [23, 32], the proximity of the storage/sedimentation basin to a fully staffed water pollution control plant may provide for optimum joint staff utilization.

Step 8 - Complete Design

The final step is to confirm that all objectives have been satisfied. This may require several iterations back through earlier steps including a reassessment of the basic criteria once the site specific costs for compliance are known.

OPERATION AND MAINTENANCE CONSIDERATIONS

The major operation and maintenance goal of downstream storage/sedimentation basins is to provide a facility that is available to its full design capacity when needed and for as long as needed. Secondary goals include clear, prompt, and complete records of performance (i.e., NPDES compliance reporting), reliability to provide for reallocation of personnel and facilities in non-storm periods, and dual use operations (such as backup treatment and/or flow equalization for dry-weather plants).

Experience has shown that frequent, periodic maintenance and equipment exercising is essential to maintain an effective readiness-to-serve. For obvious reasons, it is recommended that this maintenance be carried out on a preplanned rather than as-available basis. Staffing requirements will be unique for each facility and operation and maintenance organization. Questions to be addressed in the operations plan include:

- Will the facility activate unattended?
- What operational staffing is necessary to complete the primary function?
- What staffing is necessary to complete the auxiliary functions?
- Will the monitoring-reporting system activate unattended?
- What activities require immediate response (multishift availability) and what activities can be deferred and for how long (to conform to standard shift)?
- What emergency conditions could be encountered and how will they be addressed?
- What operational decisions must be made and who has the responsibility/control?

- What operational decisions can be implemented remote from the site and which, if any, require direct observation?

- What are the standard operating procedures to ensure safety of the public, operators, and equipment?

Recognizing that unit activations (storms) can occur at any time and generally with short warning, adequate provisions must be made for parking, assembly and briefing, and operating station access. Backup plans for automated actions should be identified including confirmation feedback and manual override if necessary.

Again, the operation and maintenance requirements and procedures should be developed from the operational plan and not from industry wide standards since there are none.

COSTS

Construction costs of downstream storage/sedimentation basins have been reported [32, Table 73] for selected demonstration facilities (including pretreatment and auxiliary systems) and are highly site specific. Adjusted to ENR 4000, the range of unit costs is from \$0.50 to \$10.00 /gal (\$0.13 to \$2.64/L) of storage capacity with a median value of about \$2.50/gal (\$0.66/L). As would be expected, facilities whose primary function is storage fall at a low end of the cost range and those which are in effect primary treatment plants rank at the high end of the range.

Storage/sedimentation basins estimated by Benjes excluding pretreatment and auxiliary systems (based on a 20 Mgal or 76,000 m^3 capacity) ranged from \$0.03/gal (\$0.01/L) for open earthen basins, to \$0.42/gal (\$0.11/L for covered concrete basins [38]. The discrepancy between Benjes estimates which are based on unit costs and hypothetical basin designs and actual construction bid costs demonstrate not only the impact of pretreatment and auxiliary systems, but the overriding importance of specific site conditions.

Another planning level cost source is a 1978 USEPA publication--Construction Costs for Municipal Wastewater Treatment Plants: 1973-1977 [39], which presents a regression analysis of construction bid costs by region, unit process, and construction component. While the emphasis is on secondary treatment, there is a good deal of potentially applicable information on preliminary treatment, influent pumping, primary sedimentation, site work, and special conditions.

Capital Cost Breakdown - Illustrative Examples

Three examples are presented in Table 24: Facilities A and B represent covered basins where the primary function is storage and Facility C represents a covered and buried facility where the primary function is sedimentation. Facility C is also unique in that it provides continuous service as a dry-weather treatment plant (22 Mgal/d or 1 m^3/s average dry-weather capacity) as well as 450 Mgal/d (20 m^3/s) peak wet-weather flow capacity. The unit cost summaries at the bottom of the table clearly demonstrate the function/cost relationship stressed in basin design: the storage units are cost-optimized on the basis of volumetric capacity and the treatment unit is cost-optimized on the basis of volume treated and discharged. The premium cost for burial of Facility C, included in the table costs, to facilitate dual use of the site above the tanks is estimated as 18% above the cost of a totally enclosed plant with exposed superstructures.

Table 24. EXAMPLE CAPITAL COST BREAKDOWNS

	Facility A [40]		Facility B [41]		Facility C [42]	
Item	Cost, $ million	% of total	Cost, $ million	% of total	Cost, $ million	% of total
General, sitework, and outside piping	3.24	91	2.1	13	27.1	29
Structural and architectural	--	--	9.6	59	32.9	35
Mechanical equipment, piping, and plumbing	0.11[a]	3[a]	3.2	19	14.3	16
Heating, ventilating, and odor control	0.08	2	0.5	3	8.4	9
Instrumentation	--	--	0.4	2	2.5	3
Electrical	0.12	4	0.6	4	7.2	8
Total	3.55	100	17.4	100	92.4	100
Cost per gallon of storage capacity[b]	0.91		0.76		6.16	
Cost per gallon treated and discharged[c]	NA[d]		0.27		0.006	

a. Equipment carried under General.

b. $/gal.

c. Capital cost divided by average annual volume discharged.

d. NA = not available.

Example 3 illustrates the use of the USEPA regression cost curves as a crosscheck for Facility C.

Operation and Maintenance Costs

As noted earlier, there are no rule-of-thumb guides for estimating operation and maintenance costs short of developing an operation and maintenance cost program for the specific facility. For planning level estimates, first-cut approximations may be developed from reported costs of operating facilities [32 (Table 73), 8] or in the case where the primary function is treatment, from standard sanitary engineering references such as the 1971 USEPA published regression curves developed by Black and Veatch [43] with adjustment to reflect intermittent operations.

EXAMPLE 3. COMPARE COST OF FACILITY IN TABLE 24 WITH EXPECTED COST OF "EQUIVALENT" PRIMARY TREATMENT PLANT USING REGRESSION CURVES FROM REFERENCE [38]

Specified Conditions

1. The design flow = 450 Mgal/d
2. The process train includes screening, grit removal, and primary sedimentation.
3. No sludge treatment is included.
4. An operations and maintenance building (with laboratory) is included.
5. Surface loading rate for Facility C is 2,700 gal/ft^2·d at 450 Mgal/d.

Assumptions

1. Design surface loading rate for conventional primary sedimentation is 900 gal/ft^2·d.
2. Primary plant component costs without sludge will be 35% of secondary with sludge.

Solution

1. Select appropriate cost curves or regression equations from reference [38].
   a. Process - Second order cost curves, page 6-54.
      (1) Preliminary treatment — $C = 5.79 \times 10^4 Q^{1.17}$
      (2) Primary sedimentation — $C = 6.94 \times 10^4 Q^{1.04}$
      (3) Laboratory/maintenance building — $C = 1.65 \times 10^5 Q^{1.02}$
   b. Construction component - second order curves, Tables 6-42 through 6-50 inclusive.
      (1) Mobilization — $C = 4.77 \times 10^4 Q^{1.15}$
      (2) Sitework including excavation — $C = 1.71 \times 10^5 Q^{1.17}$
      (3) Pilings, special foundation, dewatering — $C = 3.68 \times 10^4 Q^{1.12}$
      (4) Electrical — $C = 1.36 \times 10^5 Q^{1.00}$
      (5) Heating, ventilating, and air conditioning — $C = 3.10 \times 10^4 Q^{1.24}$
      (6) Controls and instrumentation — $C = 5.06 \times 10^4 Q^{1.12}$
      (7) Yard piping — $C = 9.96 \times 10^4 Q^{1.03}$
2. Select "equivalent" design flow for conventional plant.
   a. Equate on basis of surface loading rates.
      $Q = (900/2{,}700) \times 450 = 150$ Mgal/d
   b. This falls outside range of sample data, use Q = 100 Mgal/d, which is upper limit of sample data.
3. Compute regional and time adjustment factors.
   Base costs are 2nd Quarter 1977 = EPA LCAT Index 134
   Current cost 3rd Quarter 1980 = EPA LCAT Index 181
   Regional multiplier from Table 7-1, page 7-14, for San Francisco is 1.3175
   Combined multiplier = (181/134) x 1.3175 = 1.78
4. Compute costs and compare.

Item	Cost, $ million (ENR 4000)	
	Facility C estimate	Computed survey cost
General, site work, and outside piping [Items b.(1),(2),(3), and (7) x 0.35 (for primary)]	27.1	40.2
Structural and architectural [Items a(1),(2), and 0.4 x (3)]	32.9	50.3
Mechanical equipment, piping, and plumbing [included under structural and architectural]	14.3	--
Heating, ventilating, and odor control [Item b(5)]	8.4	5.9
Instrumentation [Item b(6)]	2.5	5.4
Electrical [Item b(4)]	7.2	8.5
Total	92.4	110.3

Comment

The USEPA guide provides an effective tool for quick cost breakdown comparisons, but application becomes questionable for plant capacities greater than 50 Mgal/d.

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Section 8

INTERNATIONAL PERSPECTIVE

INTRODUCTION

The application of storage/sedimentation controls to urban stormwater problems is not unique to the United States. In fact, in this era of excellent communications and increasing technology-sharing on an international scale, basically similar approaches are found in many areas of the world. This is particularly evident in the highly productive and densely developed nations for which receiving water quality is of great concern.

Stahre, in his comprehensive manual on storage/sedimetation practices in Europe, ranks principal urban stormwater problems as: (1) flooding, (2) discharge of untreated wastewater (CSO), and (3) shock loadings of the wastewater treatment plants [1]. Similarly, Kuribayashi and Nakamura identify combined sewer problems in Japan as threefold: (1) flooding, (2) pollution as a result of excess overflow, and (3) pollution by primary effluent [2]. The latter is a result of limitations in secondary process treatment capacities to 1.3 to 1.5 times average dry-weather flow and the common practice of returning supernatants from sludge processing facilities to the primary units. Because of this supernatant return, excess wet-weather flows discharged from secondary plants after only primary treatment may be heavily contaminated.

Chambers and Tottle have documented the benefits of onsite detention facilities as an alternative to conventional storm sewer systems in Winnipeg, Canada [3]. The impoundments were found to be well suited to the low relief topography, and the impermeable nature of the soil made attractive wet ponds feasible and recreationally as well as technically effective. Source controls emphasizing infiltration and percolation, although introduced only in the early 1970s, now number several hundred installations in Sweden, according to Stahre.

In the United Kingdom, traditional design of combined sewers has been to provide sewer capacity equivalent to six times the average dry-weather flow. Excess flows are allowed to overflow directly to the receiving water. Wastewater treatment plants are designed to provide biological treatment to 3.0 times dry-weather flow. Flows in the range of three to six times dry-weather flow are treated in storage/sedimentation tanks, sized to provide a minimum of 2 hours detention before overflowing to the receiving water. This system is particularly effective in the United Kingdom because of the uniformity and low intensity of its rainfall.

In West Germany, where many of the cities are also served by combined sewers, the climate, topography, and sewer catchment configuration often combine to produce a pronounced first flush effect. A series of dispersed upstream storage basins with simple diversions and flowrate controls was found to offer a promising solution for water quality protection. Detailed guidelines have been prepared by the state agencies to cover the planning, design, and operation of these and alternative facilities [4]. Where accommodated by existing hydraulics, many basins have been designed to facilitate self cleaning.

In Japan, Kuribayashi and Nakamura conclude that onsite and offsite storage of stormwater (70% if the sewered areas are served by combined sewers) and the bleeding of it back to the treatment works during low dry-weather flow periods seems to be one of the most feasible and effective solutions. They note that because the storage of stormwater can solve pollution problems caused by CSO as well as flooding, this measure is gradually becoming accepted by many city engineers.

In each of the above examples, hydrology, topography, and existing facilities and practices have been important factors in determining the direction of cost-effective approaches.

Typical of the international urban stormwater runoff and combined sewer overflow control and treatment techniques and practices are the following European examples of a storage/sedimentation practices manual, several flow control devices, and two innovative technology applications presented in this section.

STORAGE/SEDIMENTATION PRACTICES MANUAL

Despite the fact that detention basins have been in use for a long time in many countries, the authorities in Sweden began to accept such faciltiies as an adequate alternative to sewer separation only in recent years. A review of the storage/sedimentation practices in Sweden along with a detailed analysis of the technical configuration, design, and layout of various storage/ sedimentation arrangements was prepared by Dr. Peter Stahre [1]. The book, directed toward municipal water and sewer engineers and consulting engineers, includes four parts: (1) systemization of facilities with respect to technical configuration and placement within the sewerage system, (2) regulation of flow from storage/sedimentation facilities, (3) design of facilities with respect to the primary function of the facility, and (4) planning requirements and cost-effectiveness analysis for storage/ sedimentation facilities. The book can also be of benefit to researchers and government workers who deal with stormwater and combined sewer overflow problems. The content of each part of the book is stuctured so that it is possible to go directly to the section of interest. Also included in the book are the results and evaluations of several hydraulic modeling tests on flow control devices.

FLOW CONTROL DEVICES

For certain cases, the flow from storage/sedimentation facilities can be controlled by means of specially designed flow regulators. These provide more effective regulation of the flow than can be accomplished with a fixed throttle section. Four different regulation arrangements are described briefly:

- Flow regulator
- Hydrobrake
- Wirbeldrossel
- Flow valve

Although the arrangements operate somewhat differently, a common feature is that they are completely self-regulating and require no special exterior control equipment

Flow Regulator

The Steinscruv flow regulator for temporarily impounding flow in the pipelines upstream of the regulator was developed by Stein in Sweden in the mid-1970s. The flow regulator consists of a stationary, anchored screw-shaped plate that is turned through 270° installed in a pipe, as shown in Figure 32. In that part of the plate which fits against the bottom of the pipeline, there is an opening to release a certain specified base flow. The opening is sized so that the flow that passes through the regulator is sufficient to maintain the self-cleaning velocity for the pipeline. The length of the flow regulator is approximately three times the diameter of the pipeline.

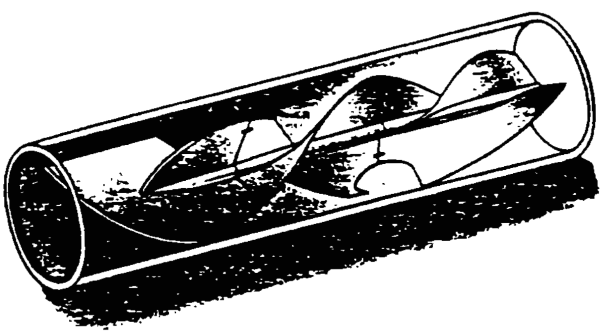

Figure 32. Flow regulator [5].

Damming takes place when the inflow to the regulator exceeds the capacity of the base opening. The extent of the damming and the volume detained are dependent on the slope of the pipe. When the flow depth reaches the crown of the pipe, the flow capacity becomes practically equal to the unregulated capacity as shown in Figure 33. It is possible to further regulate the flow by using several flow regulators in series.

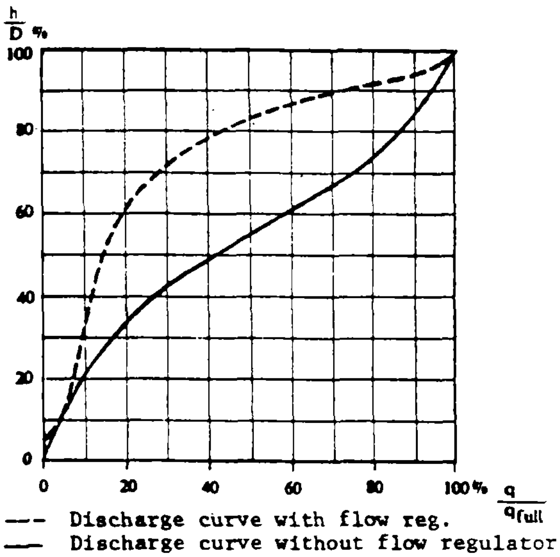

Figure 33. Comparison of discharge curves for unrestricted pipe and pipe with flow regulator [5].

The flow regulator can be used in either separate storm sewers or combined sewers to control storage. However, to prevent clogging of the regulator by debris, a diameter of 30 in. (800 mm) has been reported as the suitable minimum dimension [1].

Hydrobrake

The Hydrobrake, developed in Denmark in the mid-1960s, is used to control outflow from a storage structure.

The Hydrobrake consists of an eccentric vertical cylindrical housing with an inlet opening located on the surface of the cylinder and an axial outlet pipe located on one base of the cylinder (see Figure 34). The Hydrobrake is installed within the storage structure so that the axial outlet pipe discharges into a downstream pipe or other conveyance structure. Several different configuations are available depending on the specific application required.

When the water level rises in the storage structure, hydrostatic pressure sets the water in motion in the Hydrobrake, as shown in Figure 35. Since the outlet pipe opening is perpendicular to the direction of rotation of the water, the flow tends to assume a helical motion and the discharge is significantly less than if the flow had taken place through a fixed throttled section. A comparison of the discharge from a Hydrobrake with a circular pipe of the same size is shown in Figure 36.

Hydrobrakes are in use presently in the United States, Canada, and Sweden.

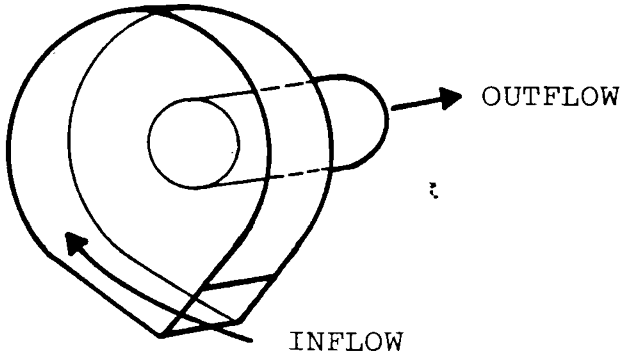

Figure 34. Schematic of a Hydrobrake [1].

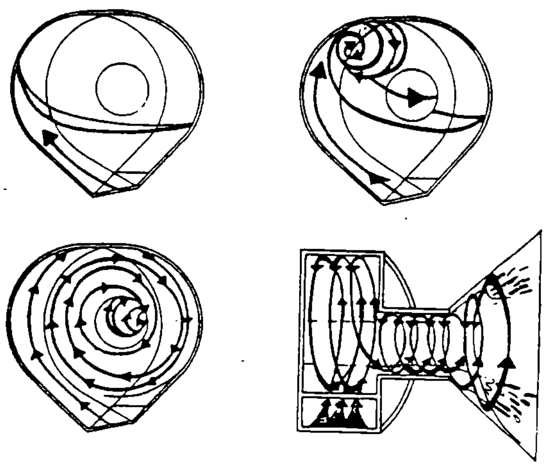

Figure 35. Schematic of flow patterns during Hydrobrake operation [6].

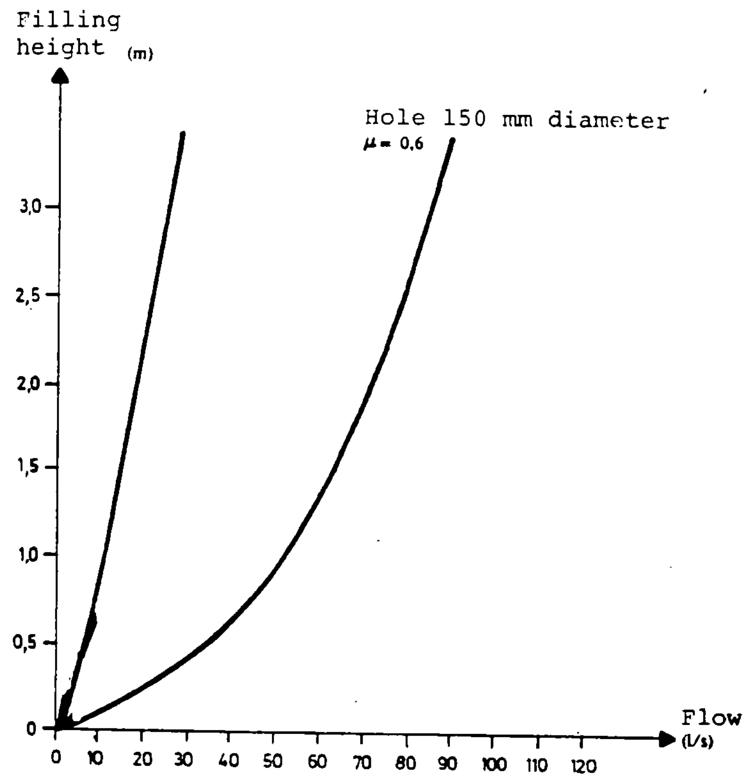

Figure 36. Discharge curve comparison for Hydrobrake and short pipe of same diameter [7].

Wirbeldrossel

The Wirbeldrossel, or turbulent throttle, developed in Germany in the mid-1970s, is another means for regulating outflow from a storage facility. In many respects, it is similar to a horizontal Hydrobrake but located immediately downstream of the storage facility (see Figure 37). The Wirbeldrossel is made up of a symmetrical cylinder having a tangential inlet and a circular outlet on the base of the cylinder. An aeration pipe is provided on the top of the unit.

A similar unit, the Wirbenventil or turbulence valve, was later developed for applications where there is a continuous base flow through the storage unit. The construction is essentially the same as for the Wirbeldrossel but consists of an obliquely positioned rotational chamber that does not require as great a headloss.

The Wirbeldrossel functions basically the same as a Hydrobrake in that rotary motion imparted to the water limits the discharge rate. The discharge is further limited by the core of air formed around the axis of rotation in the cylinder housing blocking a great part of the outlet opening.

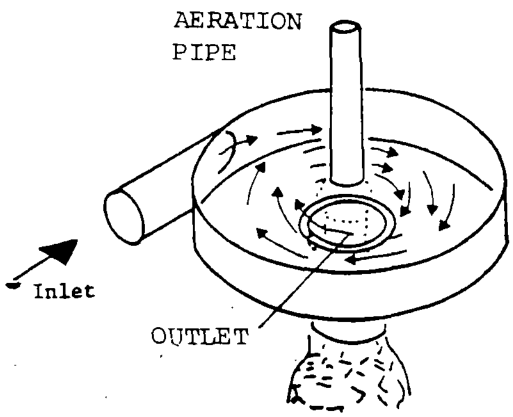

Figure 37. Schematic of flow pattern in a Wirbeldrossel [1].

The discharge is dependent on the pressure head, size of the inlet pipe, and the outlet opening. A comparison of the discharge curves for a Wirbeldrossel and a circular pipe both having the same outlet opening is shown in Figure 38.

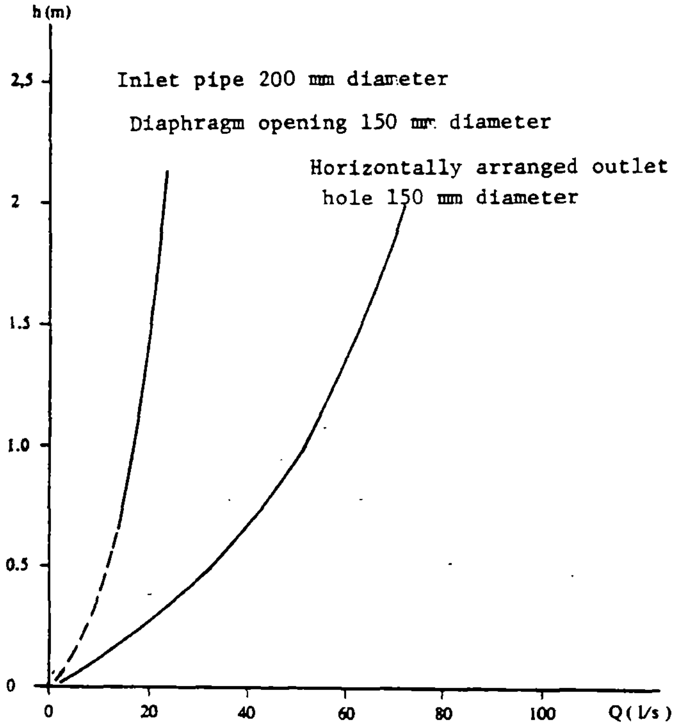

Figure 38. Discharge curves for Wirbeldrossel and circular outlet of the same size [8].

Flow Valve

The flow valve was developed in the late 1970s in Sweden as a device for holding the outflow from a detention facility constant. The flow valve is essentially a central outlet pipe surrounded by a pressure chamber filled with air as shown in Figure 39. The top part of the pressure chamber and its connection to the central outlet pipe are made of flexible rubber fabric; the rubber fabric is braced at the inlet and outlet of the center pipe.

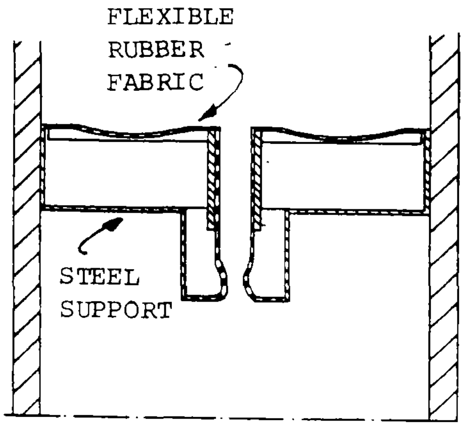

Figure 39. Diagram of a flow valve [9].

Water pressure on the upper portion of the rubber fabric is propagated through the pressure chamber displacing the fabric at the outlet section. Thus, the hydraulic capacity of the outlet is throttled by the change in outlet cross-sectional area. The resultant effect is that the discharge through the flow valve remains constant and independent of the pressure head as shown in Figure 40.

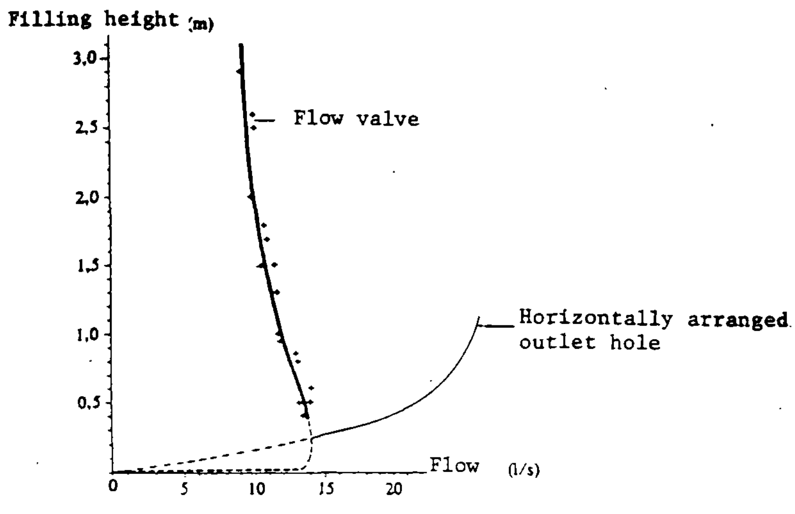

Figure 40. Typical discharge curve for flow valve [1].

INNOVATIVE TECHNOLOGY APPLICATIONS

Flow Balance System

An innovative approach to urban stormwater treatment for the protection of lakes has been developed and applied at several locations in Sweden by Karl Dunkers. The patented system uses a portion of the lake volume to store and settle urban runoff before discharge. A schematic of the system in shown in Figure 41.

A grid of wooden platforms floating on the lake's surface support flexible PVC fiber glass cloth that extends to the lake bottom and forms a series of baffled rectangular cells isolating the main body of the lake from the urban stormwater runoff. To ensure plug flow through the facility, the openings in the baffles are placed alternately at the top and bottom in adjacent cells. Stormwater runoff entering the cells displaces lake water from sequential cells. Stored runoff is withdrawn from the inlet cell and treated prior to its discharge to the receiving water. Lake water replaces the withdrawn stored runoff at the grid's outlet, so the stormwater in the grid flows back toward the inlet cell. This system is an excellent example of low cost, but effective, stormwater storage for receiving water quality protection.

An operating facility is located at Lake Trehormingen at Huddinge/Stockholm, Sweden. To prevent eutrophication of the lake resulting from phosphorus loadings, a treatment plant removes phosphorus from the stored stormwater.

Self-Cleaning Storage/Sedimentation Basin

Typically, removal of settled solids from an inline storage facility has been a problem that required an auxiliary flushing system of some sort. An example of an innovative approach to eliminating this problem is included in an inline storage/sedimentation tank in Zurich, Switzerland. A continuous dry-weather channel, which is an extension of the tank's combined sewer inlet, is formed by a number of parallel grooves connected at their end points similar to that shown in Figure 42.

The bottom groove must be given a certain slope for the water to flow by gravity through the basin. Thus, the outlet can be significantly lower than the inlet depending on the size of the basin. This approach should only be considered for small basins, less than about 132,000 gal (500 m^3) [1]. Any solids that have settled in the basin during its storage operation are resuspended by the channelized flow during the drawdown following a storm event.

For small storms, the runoff is completely captured. If the basin fills, the overflow is discharged over a weir at the end of the basin.

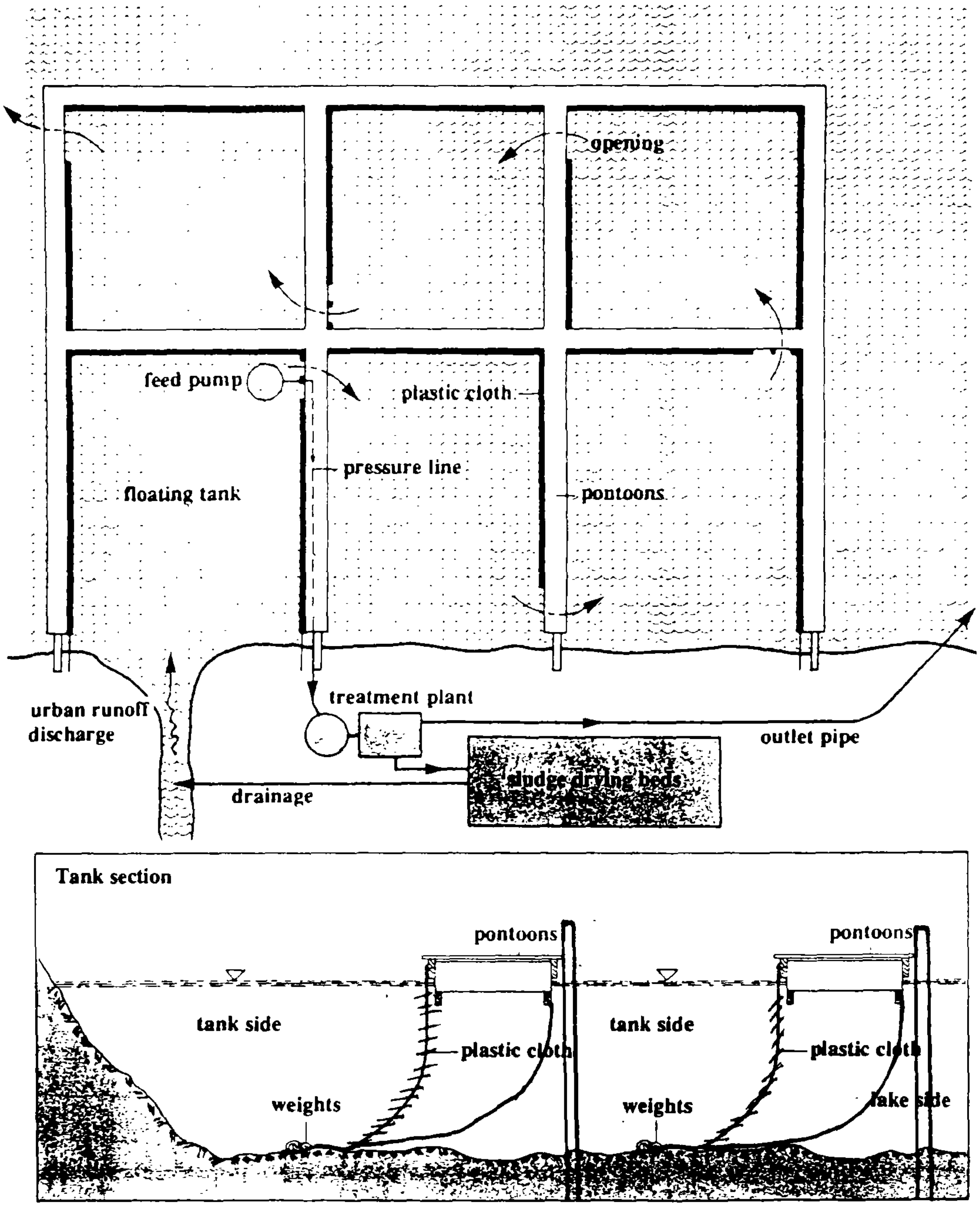

Figure 41. Schematic of pontoon tank system at Lake Tehorningen, Sweden.

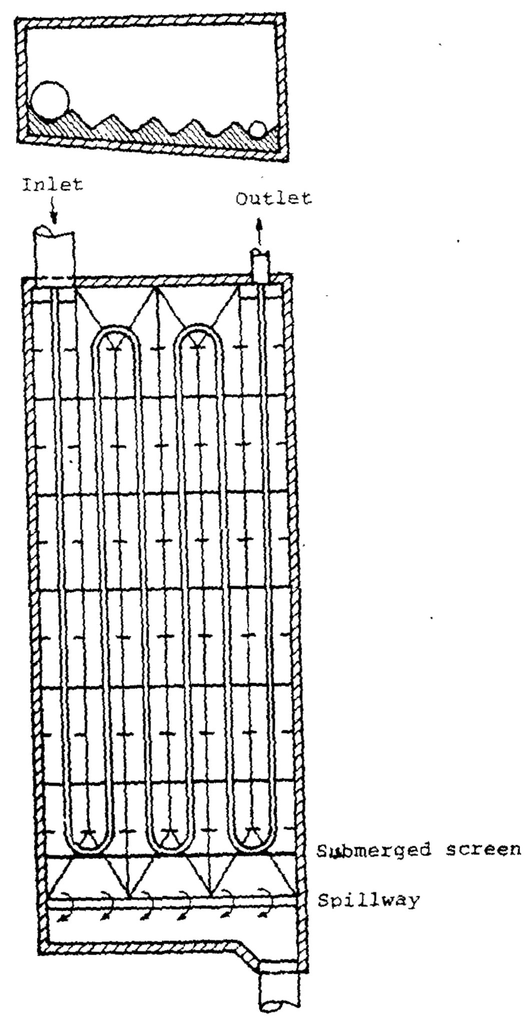

Figure 42. Self-Cleaning Storage/Sedimentation Basin used in Zurich, Switzerland [10].

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Appendix A

POLLUTANT CHARACTERIZATION AND ESTIMATION OF REMOVAL

INTRODUCTION

Characterization of the pollutants in the flow (either stormwater runoff or CSOs) is important to the estimation of the pollutant removal by storage or sedimentation or both. In this appendix, the characterization of pollutants is discussed from the standpoint of sample collection and sample analysis. The approach and methodology to be used to collect representative samples is presented. Also presented are discussions of the selection of pollutants to be analyzed, particle size determination, and the pollutant distribution versus particle size and specific gravity.

A suggested analytical method for flow routing and for pollutant routing is described. Pollutant removal can be simulated by (1) characterization by magnitude, or (2) characterization by particle size and specific gravity distribution. Both of these methods are presented along with the appropriate equations and figures.

POLLUTANT CHARACTERIZATION

Characterization studies for stormwater runoff and CSO are conducted to determine (1) the physical, biological, and chemical characteristics, and the concentration of constitutents in the wastewater; and (2) the best means of reducing the pollutant concentrations. Procedures for wastewater sampling and methods for sample analysis are described in this section.

Sample Collection

The sampling techniques used in wastewater characterization studies must assure that representative samples are obtained because the data from the analysis of the samples will ultimately serve as a basis for designing treatment facilities. Sampling programs must be individually tailored to fit each situation. Suitable sampling locations must be selected, and the frequency and type of sample to be collected must be determined since the composition of most stormwater runoff and combined sewer overflows varies considerably with time.

Sampling Locations. Sampling locations should be selected where flow conditions encourage a homogeneous mixture. In sewers and deep, narrow channels, samples should be taken from a point one-third the water depth from the bottom. The collection point in wide channels should be rotated across the channel. At all times, the flow velocity at the sample point should be sufficient to prevent deposition of solids. When collecting samples, care should be taken to avoid creating excessive turbulence that may liberate dissolved gases and yield an unrepresentative sample.

Sample Intervals. The amount of flowrate variation dictates the time interval for sampling; the interval must be short enough to provide a true representation of the flow. Even when flowrates vary only slightly, the concentration of pollutants may vary widely. Frequent sampling (10- or 15-minute uniform intervals) allows estimation of the average concentration during the sampling period.

Sampling Procedure. To adequately characterize the pollutant concentration variation with time, discrete samples must be collected. These discrete samples must be of sufficient volume so that the desired analyses can be performed. The samples can be obtained by automatic samplers or by individual grab samples. The number and size of the samples required are determined by the analyses to be performed. The physical, chemical, and biological integrity of the samples must be maintained during the interim period between sample collection and sample analysis; provision must be made for preserving the samples. Preservation techniques and maximum holding periods for some selected parameters are shown in Table A-1 [1].

Sample Analysis

Effective data analysis should include, as a minimum, the definition of (1) flow extremes (e.g., the ratio of dry-weather flow to maximum conduit capacity); (2) frequencies of occurrence of flowrates and parameter loadings; (3) types and frequencies of samples; (4) mean values and ranges of characteristics; (5) rates of change patterns and prestorm impacts; (6) site and time dependency (e.g., size of area, land use, time of day, and seasonal effects); and (7) special conditions or qualifications (e.g., construction impacts, plant bypasses, atypical flows or source area management, snowmelt versus rainfall-runoff) [3].

Careful thought should be given to the selection of the pollutants to be included in the analysis of samples. The analysis should include only those pollutants that may be affected by storage or sedimentation or both. Standard, easy to perform analyses of the physical, biological, and chemical characteristics should be included. Additional, more difficult to perform analyses should be done only if they are of specific interest.

Typically, analyses for BOD_5, SS, dissolved oxygen, total nitrogen, total phosphorus, and total coliforms are included in stormwater runoff and CSO characterizations. Less frequently incorporated analyses include various heavy metals, VSS, COD, and grease.

Where storage, sedimentation, or both are to be used as a means of treatment or control, the suspended solids (both total and VSS) become important parameters. The effectiveness of the sedimentation process depends greatly on the size and specific gravity of the particles. In addition, the effectiveness of the pollutant removal by sedimentation is determined by the pollutant distribution associated with the particle size and specific gravity distribution. Thus, for storage or sedimentation facilities, the suspended solids and particle size distribution analyses become very important to the quality characterization.

Table A-1. PRESERVATION OF WASTEWATER SAMPLES [1]

Parameter	Preservative	Maximum holding period
Acidity-alkalinity	Refrigeration at 4°C	24 h
BOD	Refrigeration at 4°C[a]	6 h
Calcium	None required	--
COD	2 mL/L H_2SO_4	7 d
Chloride	None required	--
Color	Refrigeration at 4°C	24 h
Cyanide	NaOH to pH 10	24 h
Dissolved oxygen	Determine onsite[b]	No holding
Fluoride	None required	--
Hardness	None required	--
Metals, total	5 mL/L HNO_3	6 mo
Metals, dissolved	Filtrate: 3 mL/L 1:1 HNO_3	6 mo
Nitrogen, ammonia	40 mg/L $HgCl_2$, 4°C	7 d
Nitrogen, kjeldahl	40 mg/L $HgCl_2$, 4°C	Unstable
Nitrogen, nitrate-nitrite	40 mg/L $HgCl_2$, 4°C	7 d
Oil and grease	2 mL/L H_2SO_4, 4°C	24 h
Organic carbon	2 mL/L H_2SO_4 (pH 2)	7 d
pH	None available	--
Phenolics	1.0 g $CuSO_4$ + H_3PO_4 to pH 4.0, 4°C	24 h
Phosphorus	40 mg/L $HgCl_2$, 4°C	7 d
Solids	None available	--
Specific conductance	None required	--
Sulfate	Refrigeration at 4°C	7 d
Sulfide	2 mL/L Zn acetate	7 d
Threshold odor	Refrigeration at 4°C	24 h
Turbidity	None available	--

a. Slow-freezing techniques (to -25°C) can be used for preserving samples to be analyzed for organic content.

b. For some methods of determination, 4 to 8 h preservation can be accomplished with 0.7 mL conc. H_2SO_4 and 20 mg $NaNO_3$. Refer to Standard Methods [2] for prescribed applications. (Footnote not in original reference.)

Note: $1.8(^{\circ}C) + 32 = {}^{\circ}F$
mg/L = g/m^3

FLOW AND POLLUTANT ROUTING

A description of the flow and pollutant routing in the Storage/Treatment Block of SWMM Version III is presented in this section. The approaches described for both flow and pollutant routing can be used for either a desktop analysis or a computer simulation analysis. Much of the remaining material in this appendix is adapted from Huber et al. [4].

Description

Flow and pollutants are routed through one or more storage/treatment units by several techniques. The flows into, through, and out of a unit are shown in Figure A-1. The units may be arranged in any fashion, restricted only by the requirements that inflow to the plant enters at only one unit and that the products (treated outflow, residuals, and bypass flow) from each unit not be directed to more than three units. Both wet- and dry-weather facilities may be simulated by the proper selection of unit arrangement and characteristics. Units may be modeled as having a detention capability or instantaneous throughflow. Pollutants or sludges may be represented as a simple mass or further characterized by a particle size distribution. A unit may remove pollutants (or concentrate sludges) as a function of particle size and specific gravity, detention time, incoming concentration, the removal rate of another pollutant, or a constant percentage. All flows and pollutants are assumed to be averages over a time step. This includes the input data and internal calculations [4].

Flow Routing

A storage or sedimentation unit may be modeled to handle flow in one of two ways: as a detention unit (reservoir) or a unit instantaneously passing all flow. In this report, discussion is limited only to the detention or reservoir application.

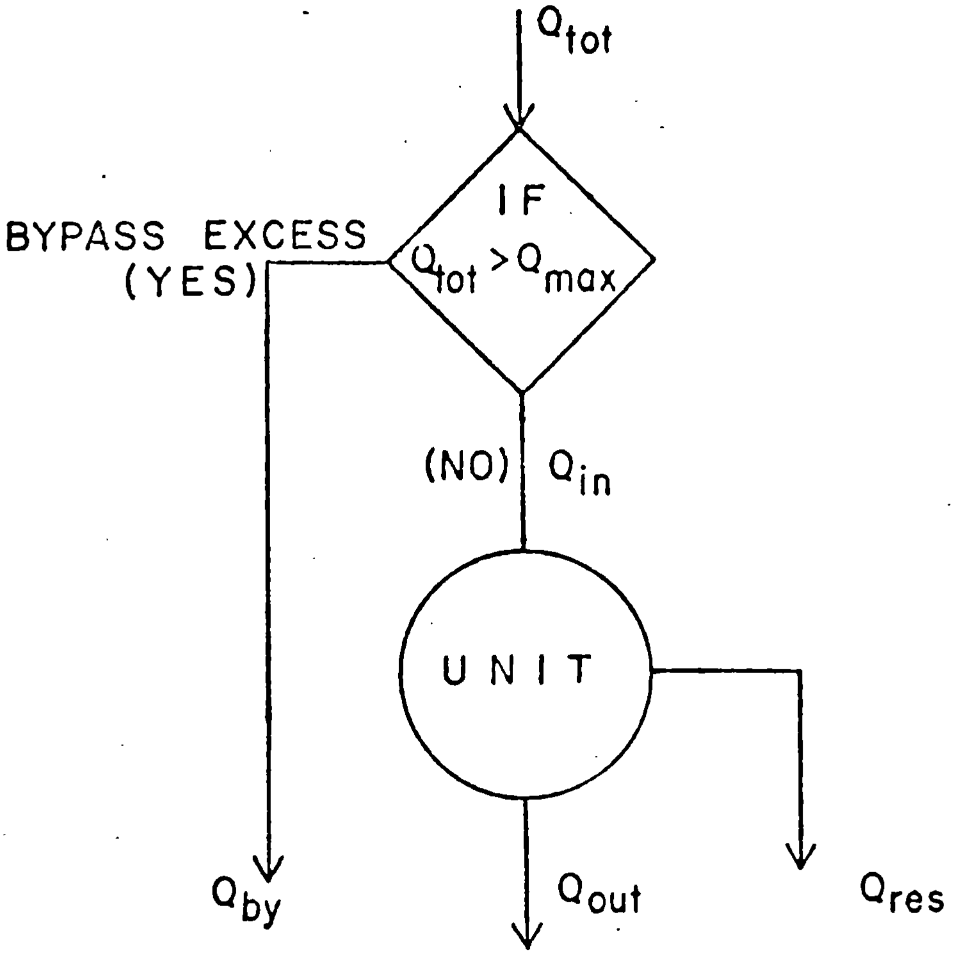
Flow routing for a simple reservoir requires that three relationships for the reservoir be known: (1) an inflow hydrograph, (2) a depth-storage relationship, and (3) a depth-discharge relationship. Routing is the solution of the storage equation which is an expression of continuity

$$\bar{I} - \bar{O} = \Delta V/\Delta t \qquad \text{(A-1)}$$

where $\bar{I}$ = average rate during Δt, ft^3/s
$\bar{O}$ = average outflow rate during Δt, ft^3/s
ΔV = reservoir volume, ft^3
Δt = time step, s

Using subscripts 1 and 2 to represent the beginning and end of the period, respectively,

$$(I_1 + I_2)/2 - (O_1 + O_2)/2 = (V_2 - V_1)/\Delta t \qquad \text{(A-2)}$$



LEGEND

Q_{tot} = TOTAL INFLOW, ft^3/s

Q_{max} = MAXIMUM ALLOWABLE INFLOW, ft^3/s

Q_{by} = BYPASSED FLOW, ft^3/s

Q_{in} = DIRECT INFLOW TO UNIT, ft^3/s

Q_{out} = TREATED OUTFLOW, ft^3/s

Q_{res} = RESIDUAL STREAM, ft^3/s

Figure A-1. Flows into, through, and out of a storage/treatment unit [4].

Equation A-2 may be transformed to

$$V_2 - V_1 = [(I_1 + I_2)/2]\Delta t - [(O_1 + O_2)/2]\Delta t \tag{A-3}$$

For a given time step, I, I_2, O_1, and V_1 are known and O_2 and V_2 must be determined. Grouping the unknowns on the left side of the equation and rearranging yields one of the two required equations:

$$0.5(O_2)\Delta t + V_2 = 0.5(I_1 + I_2)\ t - (0.5(O_1)\Delta t - V_1) \tag{A-4}$$

The second equation is found by relating O_2 and V_2, each of which is a function of the reservoir depth. The procedure is illustrated in the following example.

The data in Table A-2 give O_2 and V_2 as functions of depth over the range of depths for which outflow exists. In this hypothetical case, outflow occurs only if the reservoir depth exceeds 8.0 feet. The 11 data triplets cover the range of interest. From this information, the corresponding values of $0.5O_2\Delta t$ (outflow volume) and $0.5O_2\Delta t + V_2$ (outflow and reservoir volume) can be calculated. The depth-discharge relationship can be a composite made up of the relationships for multiple outlets.

Table A-2. ROUTING DATA FOR HYPOTHETICAL RESERVOIR

(1) n	(2) Elevation h ft	(3) Depth y ft	(4) Volume V_2 1000 ft^3	(5) Discharge O_2 ft^3/s	(6) O2DT2 $0.5O_2\Delta t$ 1000 ft^3	(7) SATERM $0.5O_2\Delta t + V_2$ 1000 ft^3
1	351.0	8.0	1,720.	0	0	1,720.
2	351.2	8.2	1,850.	10.	108.	1,958.
3	351.4	8.4	2,000.	20.	116.	2,216.
4	351.6	8.6	2,220.	35.	378.	2,598.
5	351.8	8.8	2,400.	50.	540.	2,940.
6	352.0	9.0	2,650.	65.	702.	3,352.
7	352.2	9.2	2,900.	80.	804.	3,764.
8	352.4	9.4	3,100.	105.	1,134.	4,234.
9	352.6	9.6	3,400.	130.	1,404.	4,804.
10	352.8	9.8	3,700.	165.	1,782.	5,482.
11	353.0	10.0	3,900.	200.	2,160.	6,060.

Column: (1) Counter
(2) Elevation from topographic map
(3) Depth = h - 343.0
(4) Volume measured or calculated volume
(5) Measured data or calculated from discharge formulas
(6) Calculated using O_2 (column 5), Δt = 21,600 s
(7) Calculated using column 4 and column 6

The computations procedure is summarized as follows:

1. Know values of I_1, I_2, O_1, Δt, and V_1 are substituted into the right side of Equation B-4. The result is the first value of $0.5O_2\Delta t + V_2$.

2. Knowing $(0.5O_2\Delta t + V_2)$, the value of $0.5O_2\Delta t$ is obtained by interpolation between adjacent values of the outflow volume (column 6) and the outflow and reservoir volume (column 7).

3. The values of V_2 and O_2 are determined and become the values of V_1 and O_1, respectively, in the next time step.

4. Add $0.5(I_1 + I_2)\Delta t$ to the new value of $0.5O_1\Delta t - V_1$ to get the new value of $0.5O_2\Delta t + V_2$.

5. Continue this process until all inflows have been routed [4].

This flow routing procedure has been adapted for computer simulation in SWMM-Version III but it can be applied also for hand computation or graphical methods.

Pollutant Routing

Pollutants are routed through a detention unit by one of two modes: complete mixing or plug flow.

Complete Mixing. For complete mixing, the concentration of the pollutant in the unit is assumed to be equal to the effluent concentration. The mass balance equation for an assumed well mixed, variable volume reservoir is:

$$d(VC)/dt = I(t)\,C^I(t) - O(t)\,C(t) - K\,C(t)\,V(t) \qquad \text{(A-5)}$$

where V = reservoir volume, ft^3
C^I = influent pollutant concentration, mg/L
C = effluent and reservoir pollutant concentration, mg/L
I = inflow rate, ft^3/s
O = outflow rate, ft^3/s
t = time, s
K = decay coefficient, s^{-1}

Equation A-5 may be approximated by writing the mass balance equation for the pollutant over the interval, Δt:

Change in mass in basin during Δt = Mass entering during Δt - Mass leaving during Δt - Decay during Δt

$$C_2V_2 - C_1V_1 = \frac{C_1^I\,I_1 + C_2^I\,I_2}{2}\Delta t - \frac{C_1O_1 + C_2O_2}{2}\Delta t - K\frac{C_1V_1 + C_2V_2}{2}\Delta t \qquad \text{(A-6)}$$

where subscripts 1 and 2 refer to the beginning and end of the time step, respectively.

From the flow routing procedure discussed earlier, I_1, I_2, O_1, O_2, V_1, and V_2 are known. The concentration in the reservoir at the beginning of the time step, C_1, and the influent concentrations, C_1^I and C_2^I are also known as are the decay rate, K, and the time step, Δt. Thus, the only unknown, the end of time step concentration, C_2, can be found directly by rearranging Equation B-6 to yield

$$C_2 = \frac{C_1V_1 + \frac{(C_1^I\ I_1 + C_2^I\ I_2)}{2}\Delta t - \frac{C_1O_1}{2}\Delta t - \frac{K\ C_1V_1}{2}\Delta t}{V_2(1 + \frac{K\Delta t}{2}) + \frac{O_2}{2}\Delta t} \qquad \text{(A-7)}$$

Equation A-7 is the basis for the complete mixing model of pollutant routing through a detention unit. This is applicable to small detention units with turbulent flow.

Plug Flow. The inflow during each time step, called a plug, is labeled and queued through the detention unit. There is assumed to be no transfer of pollutants between plugs. The outflow for any time step is comprised of the oldest plugs or fractions thereof or both present in the unit. This is accomplished by satisfying continuity for the present outflow volume (see previous section on Flow Routing):

$$\sum_{j=JP}^{LP} V_j \cdot f_j = V_o \qquad \text{(A-8)}$$

where V_o = volume leaving unit during the present time step, ft^3
V_j = volume entering unit during j^{th} time step (plug j), ft^3
f_j = fraction of plug j that must be removed to satisfy continuity with V_o, $0 \leq f_j \leq 1$
JP = time step number of the oldest plug in the unit
LP = time step number of the youngest plug required to satisfy continuity with V_o

The detention time (s) for each plug j is calculated as

$$(t_d)_j = (KKDT - j)\Delta t \qquad \text{(A-9)}$$

where KKDT = present time step number

For a plug j leaving the unit, the amount of pollutant leaving is

$$(P_o)_j = (P_i)_j\ f_j\ (1.0 - R_j) \qquad \text{(A-10)}$$

where $(P_o)_j$ = amount of pollutant leaving unit in plug j, lb
$(P_i)_j$ = amount of pollutant entering unit with plug j, lb
R_j = removal fraction for plug j

The manner in which R_i is calculated is decided by the user; however, as with the complete mixing option, R_i should be a function of $(t_d)_i$. The technique for developing removal equations is discussed later. The remaining pollutants in each plug leaving the unit are totaled to give the total amount discharged during the present time step.

In either the complete mixing or plug flow mode, the removed pollutants are accumulated in the unit and combined with the water drained or drawn from the unit to form the residuals stream. The water draw-off rate can be either a fraction of the remaining storage or a constant rate.

Sludge can be assumed to consist of the removed suspended solids and water drawn from the storage or sedimentation unit. Sludge should be treated simply as one of the flows leaving the unit. The residuals stream from the unit can only be termed "sludge" if suspended solids are routed.

POLLUTANT REMOVAL SIMULATION

Pollutants may be characterized by their magnitude (i.e., mass flow and concentration) or by particle size and specific gravity distributions. Describing pollutants by their particle size distribution is especially appropriate where small or large particles dominate or where several storage or sedimentation units are operated in series. For example, if the influent is primarily sand and grit, then a sedimentation unit would be very effective; if clay and silt predominate, sedimentation may be of little use. Also, if several units are operated in series, the first units will remove a certain range of particle sizes thus affecting the performance of downstream units. The pollutant removal mechanism peculiar to each characterization is discussed below.

Characterization by Magnitude

If pollutants are characterized only by their magnitude, then removal of any pollutant may be simulated as a function of detention time (in minutes, detention units only), incoming concentration, inflow rate, the removal fraction of another pollutant, the incoming concentration of another pollutant, or any combination of the above "removal factors." Two functional forms can be used to construct the desired removal equation:

$$R = \left(a_9 e^{a_1 x_1} x_2^{a_2} + a_{10} e^{a_3 x_3} x_4^{a_4} + a_{11} e^{a_5 x_5} x_6^{a_6} + a_{12} e^{a_7 x_7} x_8^{a_8}\right)^{a_{13}} \qquad \text{(A-11)}$$

or

$$R = \left(a_9 e^{(a_1 x_1 + a_2 x_2 + a_3 x_3 + a_4 x_4)} x_5^{a_5} x_6^{a_6} x_7^{a_7} x_8^{a_8}\right)^{a_{10}} \qquad \text{(A-12)}$$

where x_i = removal equation variables
a_j = coefficients
R = removal fraction $0 \leq R \leq 1.0$

The removal equation variables, x_i, may be assigned to be parameters such as detention time, flowrate, inflow concentration of the parameter being removed, inflow concentration of another parameter, etc. These are parameters that are computed at each time step. (If they are not assigned to be specific parameters, the remaining x_i are set equal to 1.0 for the duration of the simulation.) The cofficients, a_j, are directly specified by the user. There is considerable flexibility contained in these two forms, and with a judicious selection of coefficients and factors, the user can probably create the desired equation. Example applications of Equations A-11 and A-12 are given below to illustrate the procedure.

One version of the Storage/Treatment Block of SWMM employed the following removal equation for suspended solids in a sedimentation tank [5].

$$R_{SS} = R_{max}(1 - e^{-Kt_d}) \tag{A-13}$$

where R_{SS} = suspended solids removal fraction, $0 \leq R_{SS} \leq R_{max}$
R_{max} = maximum removal fraction
t_d = detention time, min
K = first order decay coefficient, L/min

This same equation could be built from Equation A-11 by setting $a_9 = R_{max}$, $a_{10} = -R_{max}$, $a_3 = -K$, $a_{13} = 1.0$, and letting x_3 = detention time, t_d.

All other coefficients, a_j, would equal zero.

Another example is taken from a study by Lager et al. [6]. Several curves for suspended solids removal from microstrainers with a variety of aperture sizes were derived. Fitting a power function to the curve representing a 35 micron microstrainer yields

$$R_{SS} = 0.0963\ SS^{0.286} \tag{A-14}$$

where R_{SS} = suspended solids removal fraction, and $0 \leq R_{SS} \leq 1.0$
SS = influent suspended solids concentration, mg/L

Equation A-12 can be used to duplicate this removal equation by setting $a_9 = 0.0963$, $a_5 = 0.286$, $a_{10} = 1.0$, and x_5 = influent suspended solids concentration, SS. All other a_j are zero.

Characterization by Particle Size and Specific Gravity Distribution

Distribution. If pollutants are characterized by their particle size and specific gravity distribution, then they are removed from the waste stream by particle settling or obstruction. Many storage or treatment processes use these physical methods to treat wastewater; sedimentation and screening are among the most obvious examples.

In this mode, pollutants are apportioned over several particle size/specific gravity ranges (e.g., 10% of the BOD is found in the range from 10 to 50 microns). Each of the selected ranges is assigned an upper and lower bound on the particle diameter and a value for specific gravity. The apportionment of pollutants over the various ranges as they enter the storage or sedimentation unit must be specified. This distribution is modified as it passes through the unit. The analysis is simplified if the particle size distribution entering the unit is assumed to remain constant over time. While the previous assumption is not usually true for actual flows, the use of flow weighted composite samples in the determination of the particle size and specific gravity distributions provides a reasonable approximation of a constant distribution.

The storage or sedimentation unit removes all or some portion of each range; the associated removal of pollutants is easily determined. For example, if a sedimentation unit removes 50% of the particles in the 50 to 100 micron range and 10% of the pollutant in question is found in this range, then 5% of the total pollutant load is removed. The total removal is determined by summing the effects of several ranges passing through the unit. Once certain particles are removed, the distribution of particle sizes for the outflow can be determined. The removed particles constitute the size distribution for the residuals stream. The next several paragraphs describe the two mechanisms available to the user for pollutant removal when pollutants are characterized by particle size.

Particle Settling. There are several forms of settling: unhindered settling by discrete particles, settling by flocculating particles, hindered settling by closely spaced particles, and compression settling within the sludge mass [7]. For simplicity, the unhindered settling of discrete particles will be the removal mechanism presented here.

The settling of discrete, nonflocculating particles can be analyzed by means of the classic laws of sedimentation formed by Newton and Stokes. Netwon's law yields the terminal particles velocity by equating the gravitational force of the particle to the frictional resistance or drag. Equating the gravitational force to the frictional drag force for spherical particles yields [8]:

$$v_s = \sqrt{(4/3)[(gd/C_D)(S_p - 1)]} \tag{A-15}$$

where v_s = terminal velocity of particles, ft/s
g = gravitational constant, 32 ft/s^2
C_D = drag coefficient
S_p = specific gravity of particle
d = diameter of particle, ft

Additionally,

$$C_D = 24/N_R, \text{ if } N_R < 0.5 \tag{A-16}$$

$$C_D = 24/N_R + 3/\sqrt{N_R} + 0.34, \text{ if } 0.5 \leq N_R < 10^4 \tag{A-17}$$

$$C_D \cong 0.4, \text{ if } N_R \geq 10^4 \tag{A-18}$$

where N_R = Reynolds number, dimensionless,

$$N_R = v_s\ d/\nu \tag{A-19}$$

and ν = kinematic viscosity, ft^2/s

A procedure for finding v_s under any of the above conditions has been demonstrated by Sonnen [9]. The average of the high and low ends of each particle size range should be used as the representative particle size in the above calculations.

A range of conditions may exist in an actual detention unit, from very quiescent, to highly turbulent and nonquiescent. Camp's [10] ideal removal efficiency, E_Q, can be used for quiescent conditions, and an adaptation of his sedimentation trap efficiency curves [10, 11, 12] as described by Chen [13] can be used to make the extension to nonquiescent conditions, as described below.

For quiescent conditions,

$$E_Q = \min \begin{cases} 1 \\ v_s/v_u \end{cases} \tag{A-20}$$

where E_Q = particle removal efficiency as a fraction $0 \leq E_Q \leq 1$
v_s = terminal velocity of particle, ft/s
v_u = overflow velocity, ft/s

Additionally,

$$v_u = Q/A = (Ay/t_d)/A = y/t_d \tag{A-21}$$

where Q = flowrate, ft^3/s
A = surface area of detention unit, ft^2
y = depth of water in unit, ft
t_d = detention time, s

Equation A-21 assumes a rectangular detention unit with vertical sides. However, a circular unit (with vertical sides) may also be modeled when characterizing pollutants by particle size. In other words, Equation A-21 is restricted to units that allow the surface area to remain constant at any depth. Applying this equation (and, thus, the entire particle size methodology) to other unit types should only be done when the surface area is independent of depth.

Equation A-20 represents an ideal quiescent basin in which all particles with settling velocities greater than v_u will be removed. Deviations from quiescent conditions can be handled explicitly based on Camp's [10] sedimentation trap efficiency curves, which were developed as a complex function of particle settling velocity, and several basin parameters, i.e.,

$$E = f(v_s y/2\varepsilon,\ v_s A/Q = v_s \ell/v_t y = v_s/v_u \qquad \text{(A-22)}$$

where E = particle removal efficiency, $0 \leq E \leq 1$
ε = vertical turbulent diffusivity or mixing coefficient, ft^2/s
v_t = flowthrough velocity of detention unit, ft/s
ℓ = travel length of detention unit, ft

Other terms are defined previously.

Camp [10] solves for the functional form of Equation A-22 assuming a uniform horizontal velocity distribution and constant diffusivity, ε. A form of the advective-diffusion equation then results in which local changes in concentration at any vertical elevation are equal to the net effect of settling from above and diffusion from below. The diffusivity will be constant if the horizontal velocity is assumed to have a parabolic distribution, (although this assumption is clearly at variance with the uniform velocity distribution assumption above). For the parabolic distribution, ε is then found from

$$\varepsilon = 0.075\ y\sqrt{\tau_0/\rho} \qquad \text{(A-23)}$$

where τ_0 = boundary shear stress, lb/ft^2
ρ = density of water = 1.94 $slug/ft^3$ (1,00 g/cm^3)

The term $\sqrt{\tau_0/\rho}$ is known as the shear velocity, u_*, and can be evaluated using Manning's equation for open channel flow [12].

$$u_* = \sqrt{\tau_0/\rho} = (v_t n\sqrt{g})/1.49\ y^{1/6} \qquad \text{(A-24)}$$

where n = Manning's roughness coefficient.

The flowthrough (horizontal) velocity, v_t, is also given by

$$v_t = \ell/t_d \qquad \text{(A-25)}$$

where ℓ = travel length of detention unit, ft
t_d = detention time, s

Equations A-23 and A-24 are then used to convert $v_s y/2\varepsilon$ to a more usable form,

$$\alpha = 0.1(v_s y/2\varepsilon) = v_s/1.5u_* = (v_s y^{1/6})/(v_t\ n\sqrt{g}) \qquad \text{(A-26)}$$

where α = turbulence factor, dimensionless when all parameters are in units of feet and seconds

Camp's sedimentation trap efficiency curves are the solution to the advective-diffusion equation mentioned previously and are shown in Figure A-2 as a function of α. Based on early work of Hazen [14] and the Bureau of Reclamation as described by Chen [13], it is assumed that an upper limit on turbulent conditions is given by $\alpha = 0.01$. Removal efficiency under these conditions is accurately represented by the function fitted to the ordinate of Figure A-2.

$$E_t = 1 - e^{-v_s/v_u} \tag{A-27}$$

or

$$E_t = 1 - e^{-(v_s t_d)/y} \tag{A-28}$$

where E_t = particle removal efficiency under turbulent conditions, $0 \leq E_t \leq 1$

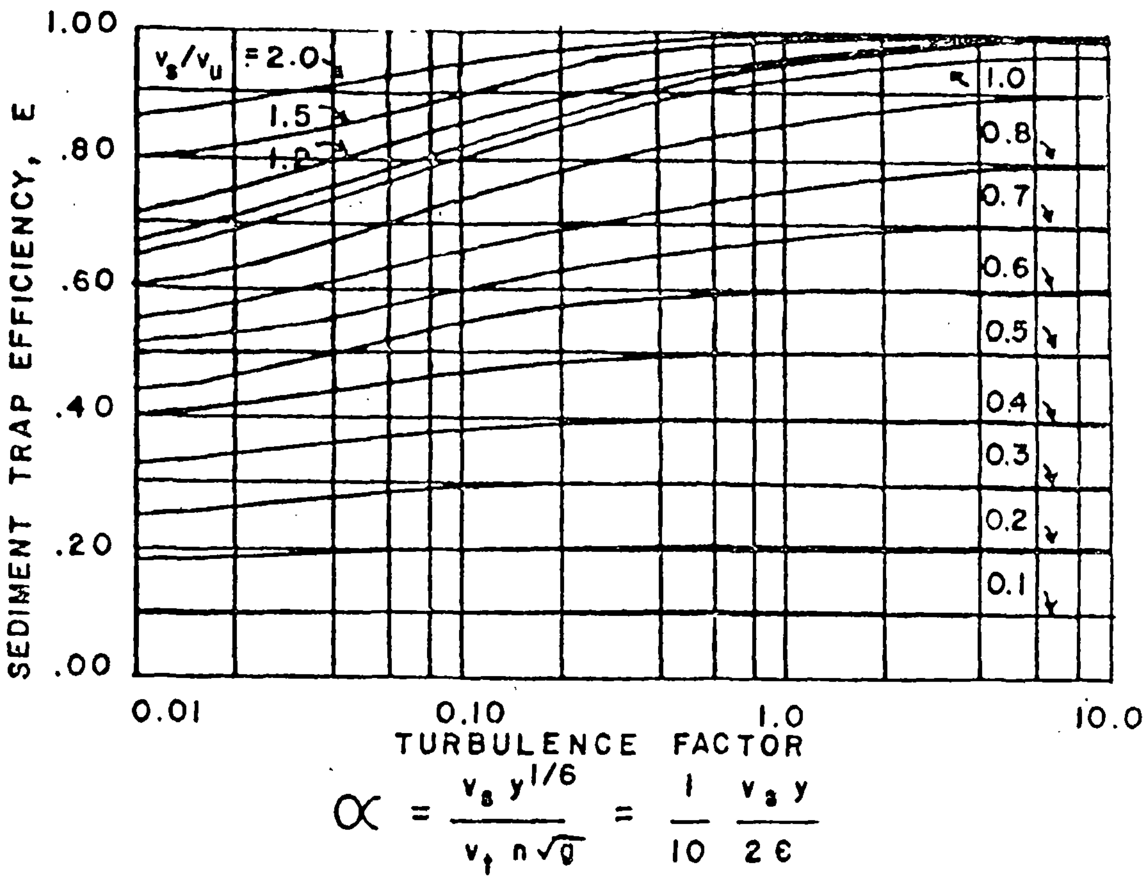


Figure A-2. Camp's sediment trap efficiency curves [12, 13]

Quiescent conditions are assumed to exist for $\alpha = 1.0$ for which removal is given by Equation A-20. Equations A-20 and A-27 are shown in Figure A-3. The parameter α may now be used as a weighting factor to obtain the overall removal efficiency, E,

$$
\begin{aligned}
E &= E_t + [(\ln \alpha - \ln 0.01)/(\ln 1 - \ln 0.01)](E_Q - E_T) \\
&= E_Q + [(\ln \alpha)/4.605](E_Q - E_t)
\end{aligned} \qquad \text{(A-29)}
$$

Thus a linear approximation (with respect to ln α) can be made of the curves shown in Figure A-2. Within reasonable accuracy, the values of the turbulence factor can be limited to $0.01 \leq \alpha \leq 1.0$.

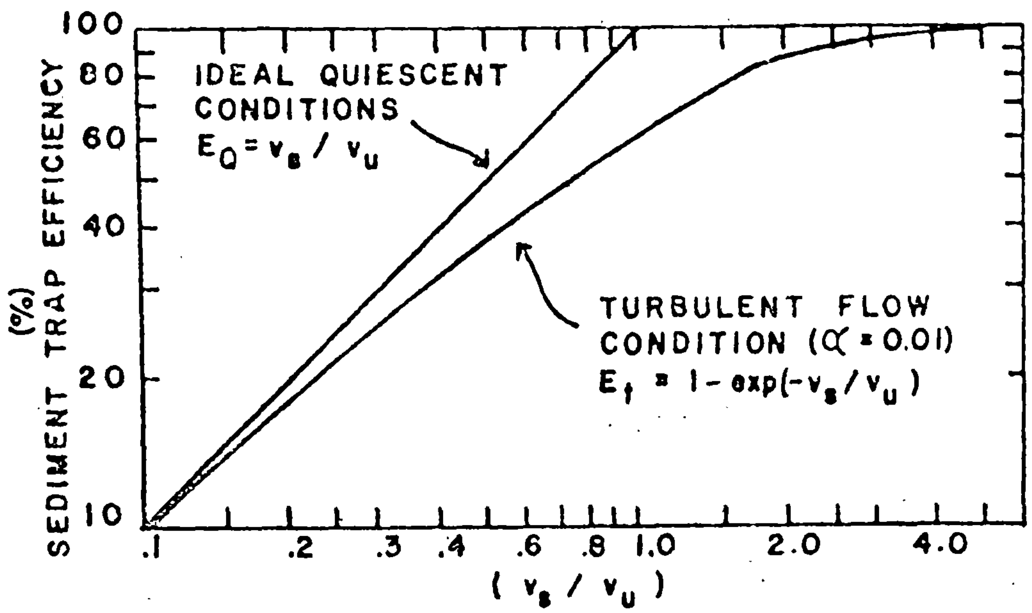


Figure A-3. Limiting cases in sediment trap efficiency.

To summarize, the particle settling computations should proceed as follows:

1. For each size and specific gravity range, a settling velocity is computed using Equations A-15 to A-19. Then for each range, all steps below are performed.

2. The turbulence factor, α, is computed from Equation A-26.

3. E_Q is computed using Equation A-20.

4. E_t is computed using Equation A-27 or A-28.

5. Finally, the removal efficiency for the particular particle size and specific gravity range is computed from Equation A-29.

In a normal simulation, several plugs leave the detention unit in any given time step. The effluent is all or part of a number of plugs depending on the required outflow as determined by the storage routing techniques discussed earlier. Thus, the effluent particle size distribution is a composite of several plugs. This composite distribution is determined by taking a weighted average (by pollutant weight in each plug) over the effluent plugs. This distribution is then routed downstream for release or further treatment. The particles that were removed from each plug are also composited and are used to characterize the residuals stream.

Comment on Characterization by Particle Size Distribution. Pollutants characterized by a particle size distribution are most easily simulated by the two removal mechanisms discussed above. The types of units that could be considered in this case would include sedimentation tanks and regular storage basins. However, these units probably represent the processes most frequently applied to the problem of combined sewer overflow and stormwater runoff. Thus, limits of the applicability of this mode are probably not too severe.

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APPENDIX B

ASSESSMENT METHODS

The relationship between rainfall and runoff is a complex and variable phenomenon, sensitive to many factors. For most stormwater analyses, the planner must model the physical system, usually with some type of mathematical model, to predict response to varying watershed conditions. A number of mathematical stormwater simulation models are available. They range in complexity from the very simple, involving desktop calculation techniques, to the highly sophisticated, involving computer simulators.

MODEL CATEGORIES

Assessment models may generally be divided into three application categories:

1. Preliminary assessment models provide indications of the existence, source, and nature of a stormwater pollutant problem. Desktop computational procedures that make use of simple equations and nomographs are usually adequate. The data requirements for these models are minimal, consisting of mean annual precipitation, watershed area, land use, population density, and sewer types. The results of preliminary assessment models are used to direct subsequent study efforts, to screen alternatives, to assess flow and quality data needs, and to aid in the selection of more sophisticated models, if necessary.

2. Continuous simulation models may help describe the variation of pollutant loadings from storm event to storm event. Variations within a single event are not described. Computer or combination desktop-computer methods generally are needed. Typically, the data requirements are long-term hourly rainfall records, overflow structure characteristics, treatment rates and storage volumes, actual overflow quantity and quality data, and varying details of drainage area characteristics, streamflow data, etc. Continuous simulation models are used to assess the magnitude of water quality problems and to evaluate alternative solutions.

3. Single event simulation models are sophisticated computer models that can be used to describe temporal and spatial variations in runoff quantity and quality within a single storm event. They require extensive and detailed data specific to the watershed, including rainfall records and runoff hydrographs, catchment and

transport system details, system maintenance information, and dry-weather and combined flow characteristics. Results obtained with these models are used in detailed planning and in the design of facilities.

Operational models, a fourth major stormwater management model category, are not assessment models, but provide real-time control of sewerage systems based on telemetered rainfall and sewer system data.

The categories of assessment models are illustrated in Figure B-1. The categories tend to blend into one another on an ascending scale of complexity and detail. More than 30 models are available. A number of the more commonly used assessment models, listed by category in order of increasing complexity are presented in Table B-1. The basic capabilities of each model are also shown. More detailed and up-to-date descriptions of capabilities and data requirements may be obtained from reports comparing and summarizing the application of such models [1a, 1b] or from the documentation or current user's guides for each model.

MODEL SELECTION

A mathematical model may be a useful tool in analyzing stormwater problems and possible solutions. The tool is most useful when carefully matched to the specific study needs. The important considerations are the level of the assessment required to meet the study objectives, the availability of the input data required, the usefulness of the model output, and the overall cost of the model. The important criteria to be considered when selecting a model are presented in Table B-2. The criteria are based on a method developed by Systems Control, Inc., for selection of water quality models [2].

In most cases, a sophisticated model should be employed only after its use has been justified by results from a simpler one. The first step in model selection is to perform a preliminary assessment of the problem. The results can be used to assess whether the water quality problems are storm-generated, or if other point and nonpoint discharges are the source. A preliminary assessment will also help to identify the pollutants and the time period of concern, the need for and appropriate level of subsequent models, and the data requirements of the study.

If justified, a list of models that might be used is then assembled. The models listed in Table B-1 are a starting point. Additional models may be identified through current publications. Adequate details of model capabilities can be obtained from the literature so that a preliminary screening can be performed. Models appropriate to the level of the study and capable of simulating the pollutants and time period of interest are given further consideration. A detailed description of the capabilities of each of these models should be obtained from the model originator. Each candidate model should then be evaluated according to the criteria listed in Table B-2.

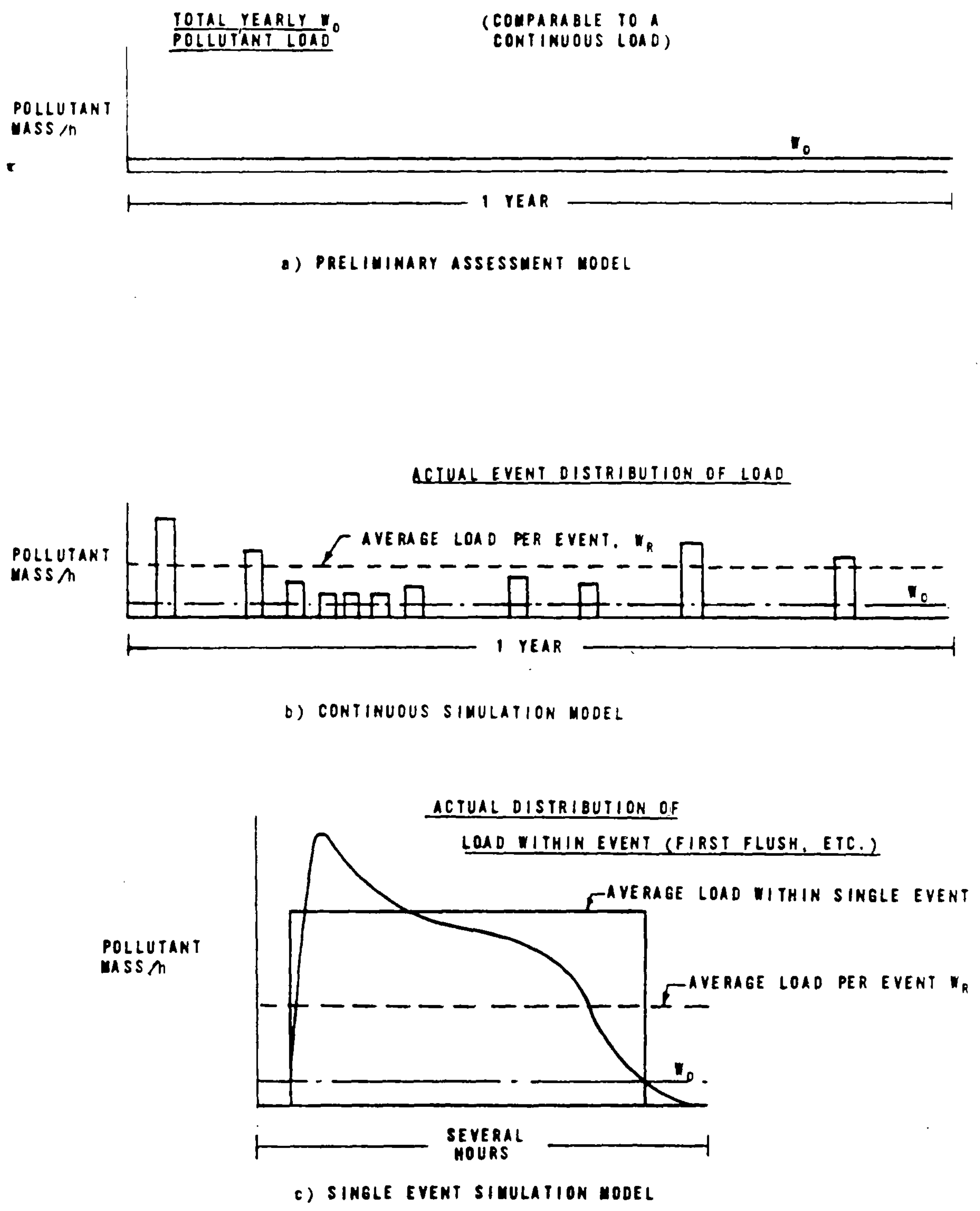

Figure B-1. Assessment model categories.

Table B-1. CHARACTERISTICS OF ASSESSMENT MODELS, BY LEVEL, IN ORDER OF INCREASING COMPLEXITY [1]

Model		Catchment hydrology						Sewer hydraulics						Wastewater quality							Miscellaneous				
Originator	Acronym	Multiple catchment inflows	Dry-weather flow	Input of several hyetographs	Snowmelt	Runoff from impervious areas	Runoff from pervious areas	Flow routing in sewers	Upstream and downstream flow control	Surcharging and pressure flow	Diversions	Pumping stations	Storage	Dry-weather quality	Stormwater quality	Quality routing	Sedimentation and scour	Quality reactions	Wastewater treatment	Receiving water quality simulation	Receiving water flow simulation	Can choose time interval	Design computations	Applied to real problems	Computer program available
Level 1 Desktop																									
University of Florida	SWMM Level 1	-	X	-	-	X	X	-	-	-	-	-	X	X	X	-	-	-	X	-	-	-	X	-	-
URS Research Company	--	-	-	-	-	X	-	X	-	-	-	-	-	-	X	X	-	-	-	-	-	-	-	-	-
EPA-MERL	--	-	-	-	-	X	X	-	-	-	-	-	X	X	X	-	-	X	X	X	X	-	X	X	-
Level 2 Continuous																									
Metcalf & Eddy	Simplified SWMM	X	-	-	X	X	X	-	-	-	-	-	X	-	X	-	-	-	X	X	X	X	-	X	X
Metcalf & Eddy	MAC	X	X	-	-	X	X	-	-	-	-	X	X	X	X	X	-	-	X	X	X	X	-	X	X
Corps of Engineers	STORM	-	-	-	X	X	X	-	-	-	X	-	X	-	X	-	-	X	-	-	-	-	-	X	X
Hydrocomp	HSP	X	X	X	X	X	X	X	X	-	X	-	X	X	X	X	-	X	-	X	X	X	-	X	-
Dorsch Consult	QQS	X	X	X	-	X	X	X	X	X	X	X	X	X	X	X	-	-	X	X	X	X	-	X	-
Level 3 Single event																									
MIT Resource Analysis	MITCAT	X	X	X	X	X	X	X	-	X	X	-	X	-	-	-	-	-	-	-	X	X	-	X	-
SOGREAH	CAREDAS	X	X	-	-	X	X	X	X	X	X	X	X	-	-	X	-	-	-	X	X	X	-	X	-
Hydrocomp	HSP	X	X	X	X	X	X	X	X	-	X	-	X	X	X	-	-	X	-	X	X	X	-	X	-
Battelle Northwest	BNW	X	X	X	-	X	X	X	-	X	X	-	X	X	X	X	-	-	X	-	-	X	X	X	X
EPA	SWMM	X	X	X	-	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X
Dorsch Consult	HMV-QQS	X	X	X	-	X	X	X	X	X	X	X	X	X	X	X	-	-	X	X	X	X	-	X	-
Water Resources Engineers	STORMSEWER	X	X	X	-	X	X	X	X	X	X	X	X	X	X	X	-	X	-	X	X	X	-	X	-

a. Proprietary.

Table B-2. MODEL SELECTION CRITERIA

Applicability
What features are necessary to the analysis?
What features are desirable?
What are the capabilities of the candidate models?
Accuracy
What level of accuracy is required?
What accuracy is justified by the available data?
How appropriate to the individual situation are the representations and assumptions of each model?
Usability
Is the available model documentation sufficient?
How usable are the output form and content?
How easily can data be changed to simulate alternative conditions?
How easily can the model be modified?
What are the capabilities of the planning staff to apply the model and analyze results?
Cost
Model acquisition cost
Equipment requirements and costs
Data acquisition costa and time requirements
Manpower costs

MODEL APPLICATION

For models beyond the preliminary assessment category, application may be thought of as consisting of three steps: calibration, verification, and analysis. In the calibration step, the known watershed characteristics and hydrologic data for a selected set of storm events are input and the unknown or uncertain model parameters adjusted so that the model output corresponds to observed runoff and receiving water responses. The model is verified when a significantly different set of conditions is input, and the model again satisfactorily predicts observed system behavior. If the model prediction for the subsequent simulation is not satisfactory, then further calibration is necessary.

Data acquisition for calibration and verification usually represents a major share of the cost of single-event and many continuous simulation models. To ensure that the usefulness of model results justifies the expenditure, consideration should be given to the parameters to be monitored and to data collection techniques. An excellent guide to monitoring and sampling for runoff data is _Methodology for the Study of Urban Storm Generated Pollution and Control_, [3].

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APPENDIX C

INFILTRATION MEASUREMENT TECHNIQUES

The infiltration rate value that is required in design of stormwater percolation facilities is the long-term acceptance rate of the entire soil surface on the proposed site for the actual stormwater to be applied. The value that can be measured is only a short-term equilibrium acceptance rate for a number of particular areas within the overall site. It is strongly recommended that hydraulic tests of any type be conducted with the actual runoff whenever possible. Such practice will provide valuable information relative to possible soil-stormwater interactions that might create future operating problems. If suitable runoff is not available at the site, the ionic composition of the water used should be adjusted to correspond to that of the runoff. Even this simple step may provide useful data on the swelling of expansive clay minerals due to sodium exchange.

There are many potential techniques for measuring infiltration including basin flooding, sprinkler infiltrometers, cylinder infiltrometers, and lysimeters. The technique selected should reflect the actual method of application being considered. For design of stormwater retention facilities, the preferred techniques are basic flooding and cylinder infiltrometers. The area of land and the volume of stormwater used should be as large as practical.

Before discussing the two techniques, it should be pointed out that the standard U.S. Public Health Service (USPHS) percolation test used for establishing the size of septic tank drain fields [1] is not recommended except for very small subsurface disposal fields or beds. Comparative field studies have shown that the percolation rate from the test hole is always significantly higher than the infiltration rate as determined from the double-cylinder (also called double ring) infiltrometer test. The difference between the two techniques is of course related to the much higher percentage of lateral flow experienced with the standard percolation test. The final rates measured at four locations on a 30 acre (12 ha) site using the two techniques are compared in Table C-1. The lower coefficient of variation (defined as the standard deviation divided by the mean value, $C_v = s/M$ for the double-cylinder technique is especially significant. A plausible interpretation is that the measurement technique involved in inherently more precise than the standard percolation test.

Table C-1. COMPARISON OF INFILTRATION MEASUREMENT USING STANDARD USPHS PERCOLATION TEST AND DOUBLE-CYLINDER INFILTROMETER[a]

	Equilibrium infiltration rate, in./h	
Location	Standard USPHS percolation test	Double-cylinder infiltrometer
1	48.0	9.0
2	84.0	10.8
3	60.0	14.4
4	138.0	12.0
Mean	82.5	11.6
Standard deviation	40.0	2.3
Coefficient of variation	0.48	0.20

a. Using sandy soil free of clay.

FLOODING BASIN TECHNIQUES

Where pilot basins have been used for determination of infiltration, the plots have generally ranged from 10 ft^2 (0.9 m^2) to 0.25 acre (0.1 ha). Larger plots are provided with a border arrangement for application of the water. If the plots are filled by hose, a canvas or burlap sack over the end of the hose will minimize disturbance of the soil [2]. Although basin tests are desirable, and should be used whenever possible, there probably will not arise many opportunities to do so because of the large volumes of water needed for measurements. In at least one known instance, pilot basins of large scale (5 to 8 acres or 2 to 3.2 ha) were used to demonstrate feasibility of wastewater percolation and then were incorporated into the larger full-scale system [3].

CYLINDER INFILTROMETERS

A useful reference on cylinder infiltrometers is Haise, et al. [4]. The basic technique, as currently practiced, is to drive or jack a metal cylinder into the soil to a depth of about 6 in. (15 cm) to prevent lateral or divergent flow of water from the ring. The cylinder should be 6 to 14 in. (15 to 35 cm) in diameter and approximately 10 in. (25 cm) in length. Divergent flow is further minimized by means of a "buffer zone" surrounding the central ring. The buffer zone is commonly provided by another cylinder 16 to 30 in. (40 to 75 cm) in diameter driven to a depth of 2 to 4 in. (5 to 10 cm) and kept partially full of water during the time of infiltration measurements from the inner ring. Alternatively, a buffer zone may be provided by diking the area around the intake cylinder with low (3 to 4 in. or 7.5 to 10 cm) earthen dikes.

The quantity of water that might have to be supplied to the double-cylinder system during a test can be substantial and might be considered a limitation of the technique. For highly permeable soil, a 1,500 gal (5,680 L) tank truck might be needed to hold a day's water supply for a series of tests. The basic configuration of the equipment during a test is shown in Figure C-1.

This technique is thought to produce data that are at least representative of the vertical component of flow. In most soils, the infiltration rate will decrease throughout the test and approach a steady state value asymtotically. This may require as little as 20 to 30 minutes in some soils and several hours in others. The test cannot be terminated until the steady state is attained or else the results are meaningless.

The following precautions concerning the cylinder infiltrometer test are noted.

1. If a more restrictive layer is present below the intended plane of infiltration and this layer is close enough to the intended plane to interfere, the infiltration cylinders should be embedded into this layer to ensure a conservative estimate.

2. The method of placement into the soil may be a serious limitation. Disturbance of natural structural conditions (shattering or compaction) may cause a large variation in infiltration rates between replicated runs. Also the interface between the soil and the metal cylinder may become a seepage plane, resulting in abnormally high rates. In cohesionless soils (sands and gravels), the poor bond between the soil and the cylinder may allow seepage around the cylinder and cause "piping." This can be observed easily and corrected, usually by moving a short distance to a new location and trying again. Variability of data caused by cylinder placement can largely be overcome by leaving the cylinders in place over an extended period during a series of measurements [2].

Knowledge of the ratio of the total quantity of water infiltrated to the quantity of water remaining directly beneath the cylinder is essential if one is interested only in vertical water movements. If no correction is made for lateral seepage, the measured infiltration rate in the cylinder will be well in excess of the "real" rate [5]. Several investigators have studied this problem of lateral seepage and have offered suggestions for handling it [5, 6, 7,].

As pointed out by Van Schilfgaarde [8], measurements of hydraulic conductivity on soil samples often show wide variations within a relatively small area. Hundred-fold differences are common on some sites. Assessing hydraulic capacity for a project site is especially difficult because test plots may have adequate capacity when tested as isolated portions, but may prove to have inadequate capacity after water is applied to the total area for prolonged periods. Parizek has observed that problem areas can be anticipated more readily by field study following spring thaws or prolonged periods of heavy rainfall and recharge [9]. Runoff, ponding, and near saturation conditions may be observed for brief periods at sites where drainage problems are likely to occur after extensive application begins.

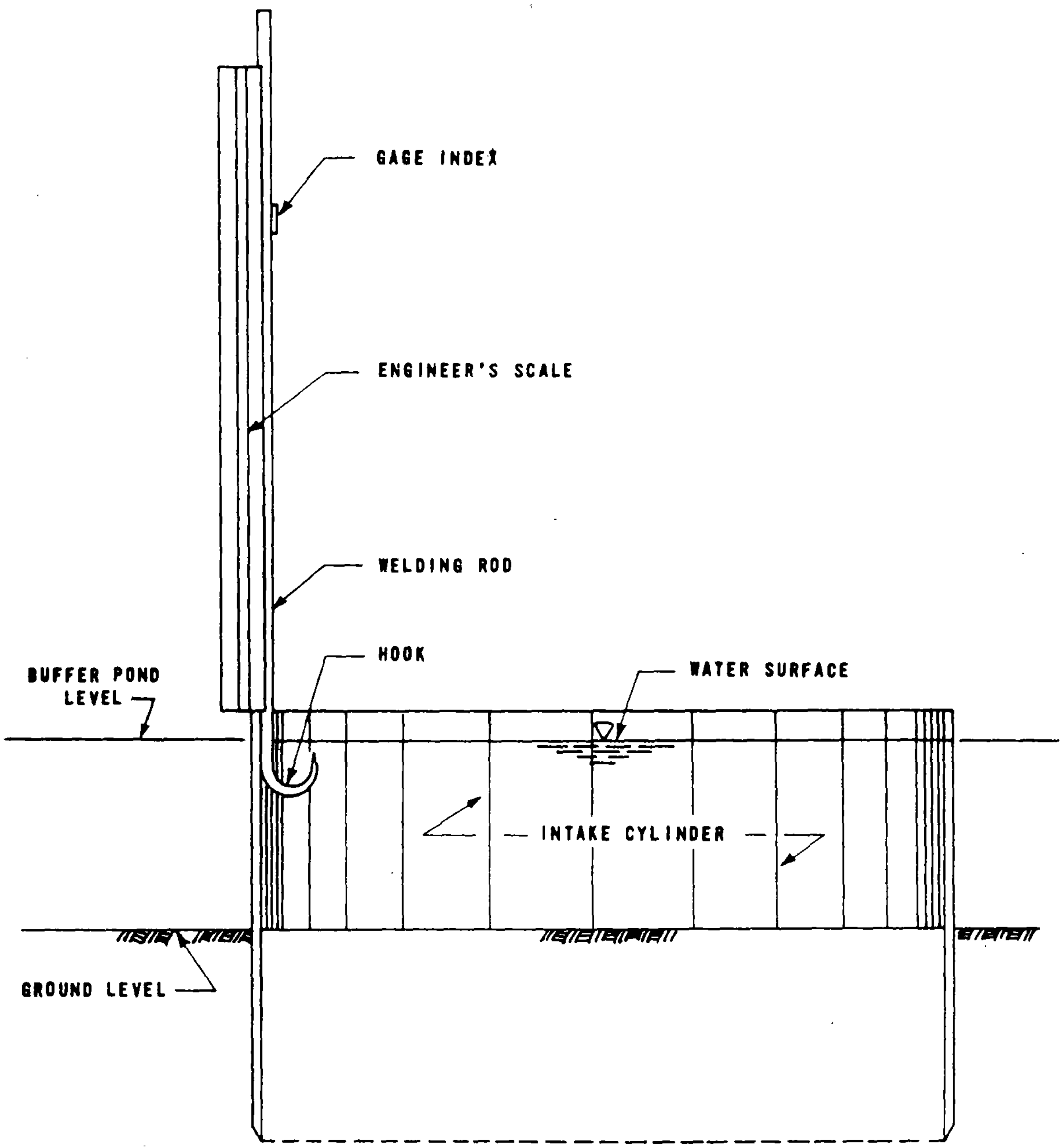

Figure C-1. Cylinder infiltrometer in use

Although far too few extensive tests have been made to gather meaningful statistical data on the cylinder infiltrometer technique, one very comprehensive study is available from which tentative conclusions can be drawn. Burgy and Luthin reported on studies of three 40 by 90 ft (12.2 x 27.4 m) plots of Yolo silt loam characterized by the absence of horizon development in the upper profile [10]. The plots were diked with levees 2 ft (0.6 m) high. Each plot was flooded to a depth of 1.5 ft (0.5 m), and the time for the water to subside to a depth of 0.5 ft (0.15 m) was noted. The plots were then allowed to drain to the approximate field capacity and a series of cylinder infiltrometer tests--357 total--were made.

Test results from the three basins located on the same homogeneous field were compared. In addition, test results from single-cylinder infiltrometers with no buffer zone were compared with those from double-cylinder infiltrometers. The inside cylinders had a 6 in. (15 cm) diameter; the outside cylinders, where used, had a 12 in. (30 cm) diameter.

For this particular soil, the presence of a buffer zone did not have a significant effect on the measured rates. Consequently, all of the data are summarized on one histogram in Figure C-2. The calculated mean of the distribution shown is 6.2 in./h (15.7 cm/h). The standard deviation is 5.1 in./h (12.9 cm/h).

Burgy and Luthin suggest that the extreme high values, while not erroneous, should be rejected in calculating the hydraulic capacity of the site. Physical inspection revealed that these values were obtained when the cylinders intersected gopher burrows or root tubes. Although these phenomena had an effect on the infiltration rate, they should not be included in the averaging process since they carried too much weight.

As a criterion for rejection, Burgy and Luthin suggest omitting all values greater than three standard deviations from the mean value. They further suggest an arbitrary selection of the mean and standard deviation for this procedure based on one's best estimate of the corrected values rather than the original calculations. From inspection of the histogram, these values might be selected as about 5 in./h (12.7 cm/h) and 3.5 in./h (8.9 cm/h), respectively. Thus, all values greater than 5.0 + 3(3.5), or 15.5 in./h (39.4 cm/h) are arbitrarily rejected: a total of 12 of the 357 tests made (3.4%).

Because it is important to provide conservative design parameters for this work, however, it is recommended that all values greater than two standard deviations from the mean be rejected. For the example, this results in the rejection of all values greater than 5.0 + 2(3.5), or 12 in./h (30.5 cm/h) from the average. A recomputation using this criterion provides a mean of 5.1 in./h (12.5 cm/h) and a standard deviation of 2.8 in./h (7.1 cm/h). This average value is within 16% of the "true" mean value of 4.4 in./h (11.2 cm/h) as measured during flooding tests of the entire plot.

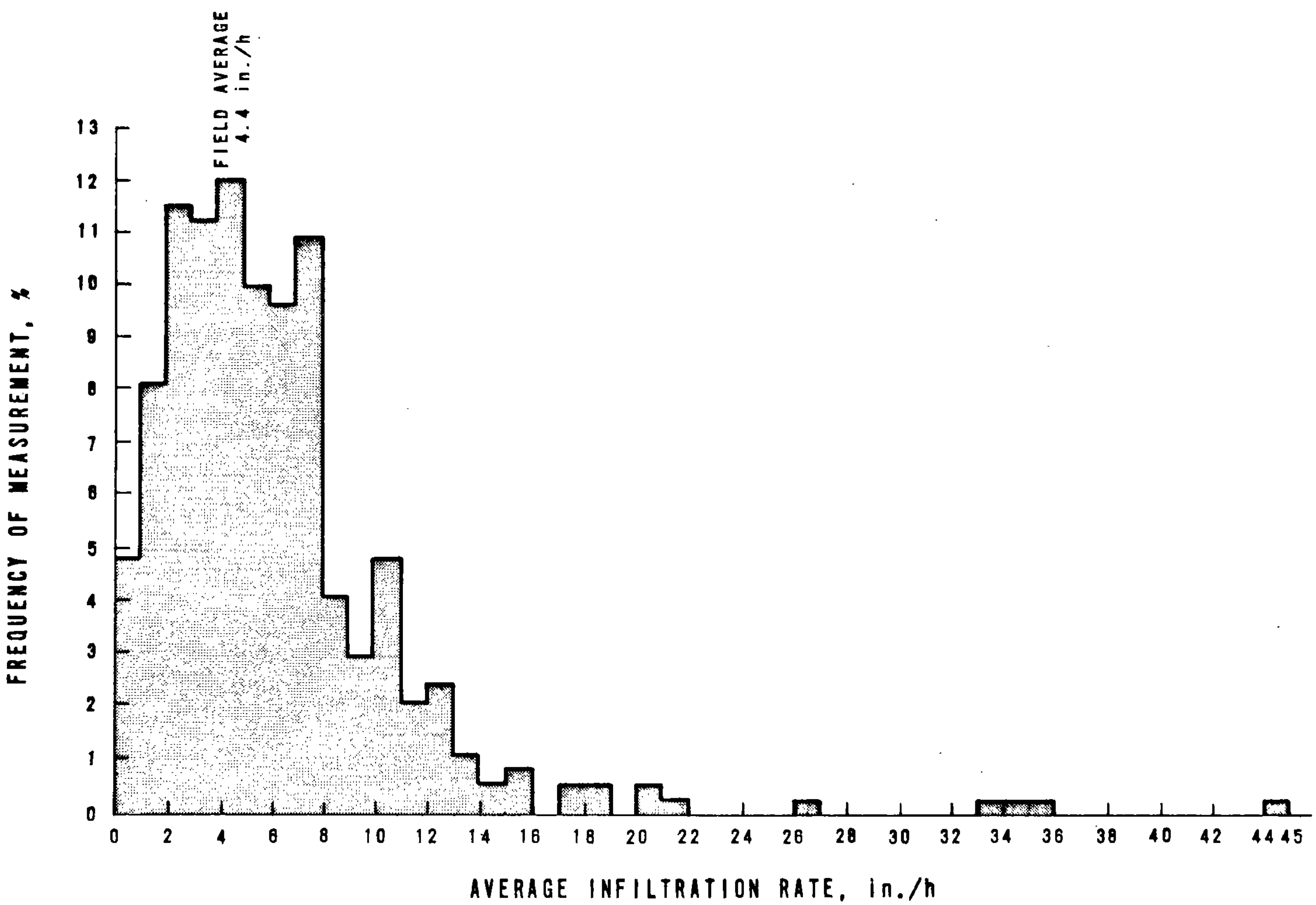

Figure C-2. Variability of infiltrometer test results on relatively homogeneous site.

The main question to be answered now is, how many individual tests must be made to obtain an average that is within some given percent of the true mean, say at the 90% confidence interval? The answer has been provided by statisticians using the Student "T" distribution. Details of the derivation are omitted here but can be found in most standard texts on statistics.

The results of two typical sets of computations are summarized in Figures C-3 and C-4. The two sets of curves are for 90% and 95% confidence intervals. The confidence interval and the desired precision are, of course, basic choices that the engineer must make. A 90% confidence in the measured mean, which is within 30% of the true mean, may be sufficient for small sites where neighboring property is available for expansion if necessary. On the other hand, 95% confidence that the measured value is within 10% of the true mean may be more appropriate for larger sites or for sites where expansion will not be easily accomplished once the project is constructed.

The coefficient of variation will have to be estimated from a few preliminary tests because it is the main plotting parameter in these figures. As an example, for the adjusted distribution of Burgy and Luthin's data with a coefficient of variation estimated at 0.55, at least 23 separate tests would be required to have 90% confidence that the computed mean would be within 20% of the true mean value of infiltration. Obviously, time and budget constraints must be considered in making the confidence and accuracy determinations; 3 to 4 man-days of work might be required to make 23 cylinder infiltrometer tests.

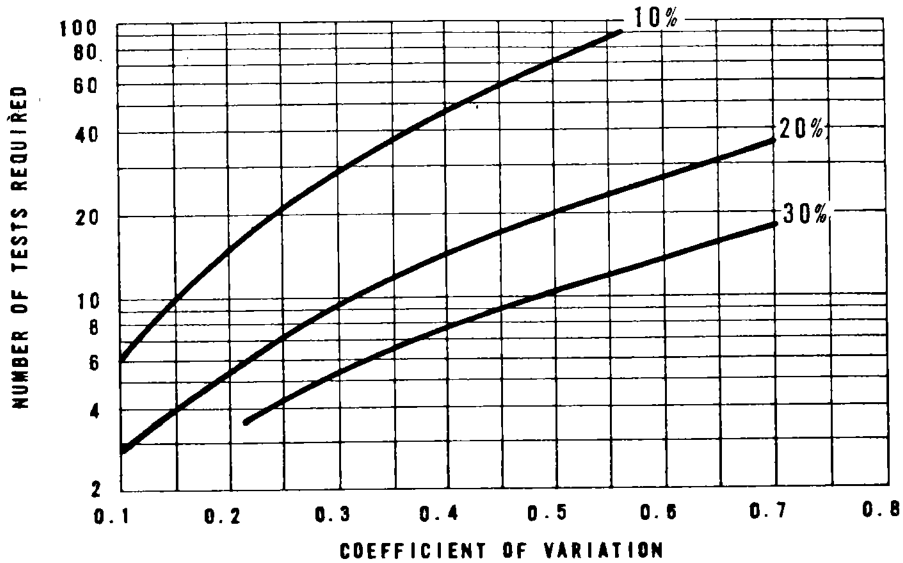

Figure C-3. Number of tests required for 90% confidence that the calculated mean is within stated percent of the true mean.

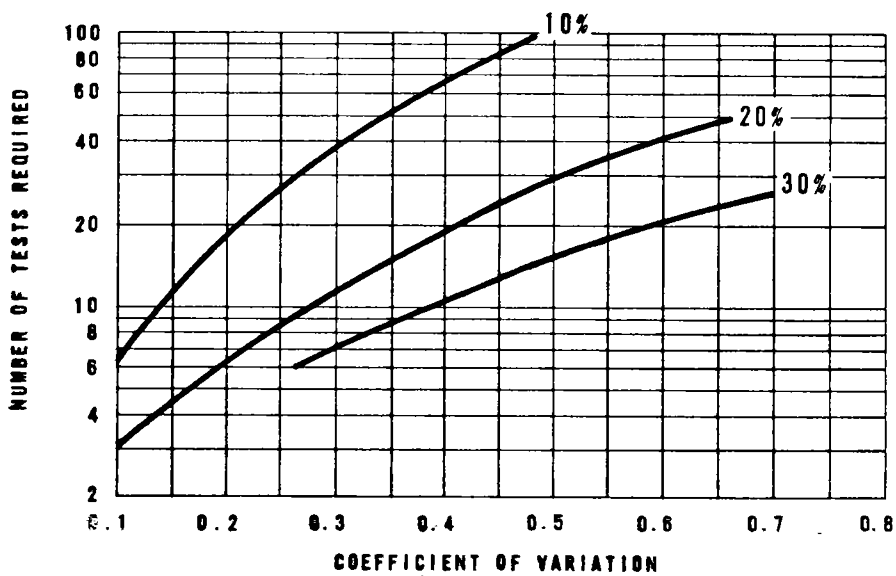

Figure C-4. Number of tests required for 95% confidence that the calculated mean is within stated percent of the true mean.

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